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Addis Ababa Science and Technology University
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SCHOOL OF ARCHITECTURE AND CIVIL ENGINEERING

POST GRADUATE STREAM

ASSESMENT OF TELECOMMUNICATION TOWERS FOR WIND AND EARTHQUAKE LOADS IN ETHIOPIA

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May, 2017

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A thesis submitted to the School of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

(Structural Stream)

Approved by Board of Examiners

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DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other university. All sources of materials used for the thesis have been duly acknowledged.

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ACKNOWLEDGMENT

I would like to gratefully acknowledge my advisor, Dr. Suresh Bora for his guidance and support throughout this thesis and the enormous effort he made to revise this document. His encouragement, support and relationship was amazing.

Next, I have no words to express my warm feeling and appreciation to my wife Mimi Hagos and friends for their lovely encouragement. Without their support I couldn't have this opportunity to complete my study. I will also extend my thanks to my mother Senait Buli for her unconditional love and prayer. Finaly I dedicate these works to my lovely daughter Efrata Mignot. Jesus will come soon.

ABSTRACT

Large number of telecommunication towers has been constructed in Ethiopia during last few decades with the rapid development of telecommunication sector in the country. These towers play a significant role especially in wireless communication and failure of such tower due to wind loads and earth quake produces magnificent loss in the economy of a country like Ethiopia. Therefore, design of telecommunication towers considering all possible extreme conditions is of utmost importance and a good design can be considered as a step towards a greater degree of sustainability. In these study the researcher tried to model one existing Telecom Tower in Ethiopia which is located in seismic zone (zone 4) of the national map using SAP2000, the results obtained from the software analysis were tabulated, compared and the governing loads are identified and conclusions were drawn. Accordingly among the two main lateral loads on the Tower (wind and Earth quack) compared by the researcher the earth quack loads govern the design.

KEY WORDS; Wind loads, Earth quack loads, Telecom Tower

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Contents

LIST OF SYMBOLS

A = gross area of Section

A_{gb} = gross area of the bearing plate

D = dead load

DC = degree of curve

E_c= modulus of elasticity of concrete

E_{ci} = modulus of elasticity of concrete at transfer

E_s= modulus of elasticity of prestressing steel

E_{si}= modulus of elasticity of steel reinforcement

ES =elastic shortening of concrete

f_{br}= concrete bearing compressive strength

f'_c= compressive strength of concrete at 28 days

f_{se} = effective stress priestess after losses

f_{sy} = yield strength of non-prestressed conventional reinforcement in tension

f^*_y = yield point stress of prestressing steel

f^*_{su} = average stress in prestressing steel at ultimate load

F = longitudinal force due to friction or shear resistance at expansion bearings

FR = friction loss

h = total depth of section

h_{min} = minimum structural depth

h_{max} = maximum structural depth

I = impact load

ICE = ice pressure

K = friction wobble coefficient

K_s = a constant for the determination of a stream pressure

l = length of prestressing steel

L = the span length in meters

LF = longitudinal force from live load

LL = live load

Mi= primary moment

ABBREVATIONS

CDMA = Code Division Multiple Access

GSM = Global System for Mobile

WAP= Wireless Web Access

NBCC = National Building Code of Canada

LRFD = Load and Resistance Factor Design

BS = British Standards

US NEHRP = United States National Earth quack Hazard Reduction Program

SRSS = Square Root of Sum of Square

CQC = Complete Quadratic Combination

API = American Petrolem Institute

MCQC = Multi-quadratic Combinations

MDF = Multi Degree of Freedom Model.

EBCS = Ethiopian Building Code.

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CHAPTER ONE

1 INTRODUCTION

1.1 Background of the Study

Telecommunication towers are a truss structure that plays a significant role in holding telecom transmission and reception antennas throughout the world. According to sources from Ethio-telecom the self-supporting antenna towers were introduced during Emperor Haile Silase in Addis Ababa in 1962 EC, then on wards on the introduction of CDMA,GSM, WAP,NGN and mobile cell phones these antennas Towers become essential item.

A number of telecommunication towers with two structural forms are available in this country and almost all of these towers have been designed only considering wind loading, since Ethiopia was considered as a country where earthquake is not frequently happening. However, with recent significant records in southern parts of Ethiopia (Hawasa) and 4.3 Richter scale around Debrebrihan, the probability of occurrence of earthquakes in the country is highlighted and design of buildings and other structures considering seismic effects is also emphasized.

According to Ethio-Telecom all detailed design drawings of towers constructed in Ethiopia were designed by the Chinese phone company ZTE up on which no earth quakes effect is considered, but since the tower members are imported from abroad its cost increases from time to time and failure of towers mean it incurs a negative socio-economic benefit to the country, due to these design of members for all possible failures is becoming important.

A failure of a telecommunication tower especially during a disaster is a major concern in two ways. Failure of telecommunication systems due to collapse of a tower in a disaster situation causes a major setback for rescue and other essential operations. Also, a failure of tower will itself cause a considerable economic loss as well as possible damages to human lives. Hence, analysis of telecommunication towers considering all possible extreme conditions is of utmost importance. (A.M.L.N. Gunathilaka)

The main objective of this research is assessing the performance of exiting towers towards wind loads and earth quack loads (which were not initially designed considering earthquake loading specially located in earth quick zone 4 of the country) and providing a clue to the readers which load governs the design of a self-supporting lattice Tower.

However, two types of telecommunication towers with different structural forms are available in the country and this study has been limited to analysis of four legged existing self-supporting lattice towers, which are the most common type of telecommunication towers in this country.

Table 1.1: Load combinations during design of Telecommunication Towers (***ANSI/TIA-222-G [1])***

Combination No.	Gravity load factor	Wind load factor	Earth Quack load factor
1	1.2	1.6	0.0
2	0.9	1.6	0.0
3	1.2	0.0	1.0
4	1.2	1.0	0.0
5	0.9	0.0	1.0

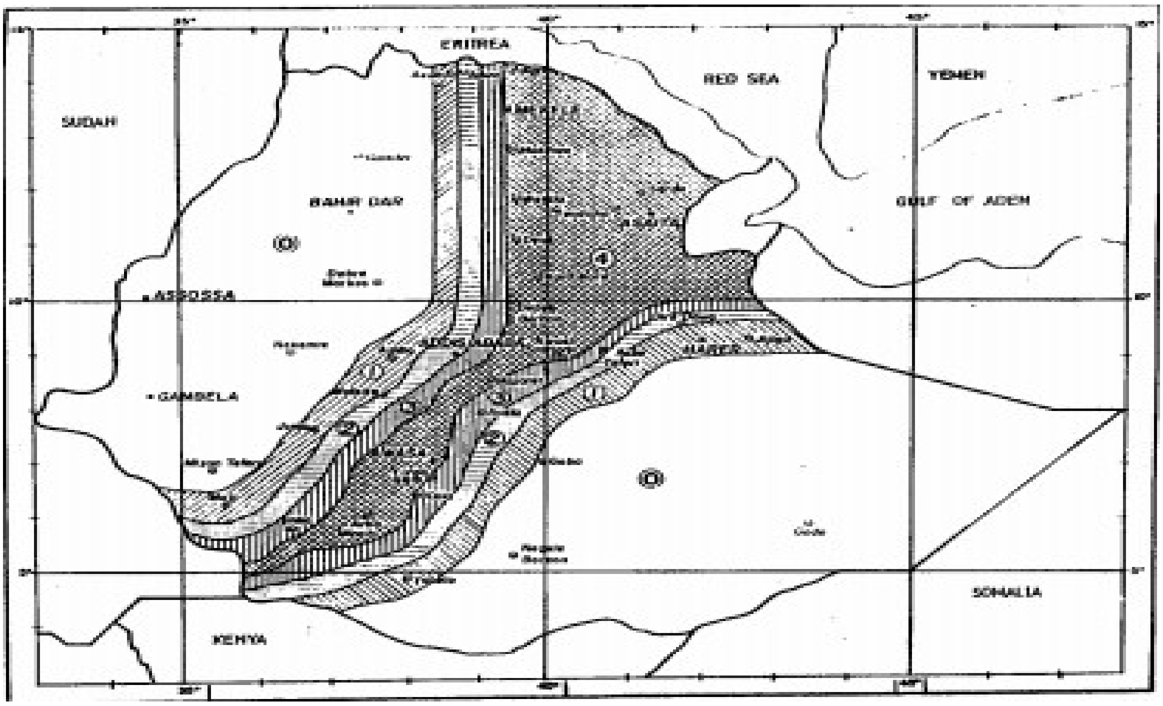


Figure 1.1 Seismic hazard map of Ethiopia (Ebcs1 -1995)

1.2 Purpose of the study

The main objective of this study is to evaluate the effects of wind and earth quack loads on the design of Telecommunication Towers. According to Ethio-Telecom so far in Ethiopia structural engineers design lattice Towers only for wind loads and ignoring the effect of earth quake even for sevier zone of the map. This philosophy arose from two main things. The first one is Ethiopia is not an Earth quake prone country so that it is un-economical to design the Towers for Earth quack. The second reasons why most structural engineers neglect Earth quake is that the Towers have strong base which can with stand the effect of lateral loads.

Recent studies emphasizes that it is important and to design structural Towers for earth quack loads together with the wind loads. According to the design combination rules it is impossible to expect both wind and earthquake loads happening simultaneously but it is better to know and design the governing load which can govern the design.

Therefor this study gives a good understanding on the design of lattice towers considering all extreme conditions which governs the design from the two main loads.

1.3 Scope of the Study

This thesis encompasses loads on the towers which can govern during the design of Telecom Towers. The study is only limited to the four legged lattice towers which is located in Ethiopia. There are a number of loads which cause failure to the structure such as ice loads temperature effects etc., but the study only limited to wind loads and earth quack loads.

1.4 Organization of Thesis

The study comprises a four legged lattice tower located in southern part of Ethiopia in Ejere Town, the tower is designed and modeled in sap2000 for both earth quake and wind loads, since the tower is located in zone four of the national map 0.1g ground acceleration is used for the seismic analysis of the tower and 22m/s wind speed is also taken for analysis of wind loads. The tower is analyzed as per the new code book of Ethiopia EBCS EN 1991-1-4. The thesis is organized in different sections which are arranged as follows: It has six chapters, which each section representing distinct phase of an overall study of lattice tower.

Chapter 1 presents general description about the thesis including; background, objectives and thesis overview.

Chapter 2 presents literature survey on details of the study by different scholars , different method of analysis to wind and earth quake effects were determined, while the response of the tower to the loads is determined and summarized.

Chapter 3 contains design example of the tower specified by the Ethio-Telecom design deteils, the example is as per the new code book EBCS EN 1991-1-4.

Chapter 4 contains coding using c sharp programming language which can calculate horizontal drift of towers up on applications of horizontal concentrated loads.

Chapter 5 contains modeling and analysis of towers using sap2000 software, results were discussed.

Chapter 6 contains conclusions and recommendations, and the last sections contain references used in this thesis work and Appendices(A,B,C)

CHAPTER TWO

2 LITERATURE REVIEW

2.1 Introduction

Latticed towers are widely used today as supporting structures, namely to support antennae in telecommunication network systems and in overhead power lines. Two types of latticed towers are generally available in today's world: guyed, towers and self- supporting towers. Self-supporting telecommunication towers are three- 'legged or four- legged space trussed structures with usual maximum height of 120m to150m. These towers consist of main legs and horizontal and transverse bracings. Main legs are usually composed of 90° angles (in four-legged towers) or 60°schifflerized angles (in three- legged towers). Different bracing patterns are used but the most common ones are the chevron and the cross bracing. Classical steel transmission towers are four-legged latticed structures. The main legs are composed of 90°angles or tubular sections. Transmission towers have several cross-arms to support the conductors and ground wires. The bracing patterns used in these towers are similar to those used in self-supporting telecommunication lattice towers.

Self-supporting telecommunication towers are usually designed under the effect of wind and ice loads, without considering earthquakes. These towers may be very important structures in a telecommunication network and the designers hold insure that they will perform well in a severe earthquake event, especially for towers located in high risk seismic areas. In the 1994 edition of CAN/CSA-S37Antennas, Towers and Antenna- Supporting Structures (CSA 1994) a new appendix was introduced to address the issue of seismic analysis of self-supporting telecommunication towers. In this appendix it is recommended that, whenever necessary, the tower should be analyzed under the effect of earthquake loading using modal superposition.The base acceleration should be compatible with the values prescribed by the National Building Code of Canada (NBCC1995) for the tower site. This recommendation is very general, and the designer is left without any specific guidance to assess whether or not a detailed analysis is truly necessary. It would therefore be desirable to rely on a simplified, quasi-static method of analysis to get an estimate of the relative importance of the seismic response of the tower. If the accuracy of such a method can be proven, detailed dynamic analysis may even become unnecessary in the majority of the cases. The objective for the designers is then to compare the effect of earthquake inertia loads to that of extreme wind loads or combined wind and ice loads.

The main design loads in the case of transmission towers are environmental loads including wind and ice or a combination of both. Several extreme cases of loading such as conductor breakage and ice-shedding are usually considered during the design process using equivalent static loads or quasi-static methods. However, earthquake effects are not considered in the design even in high risk seismic areas. There are reports (Pierre, 1995 and Kempner, 1996) of damages in some transmission towers during recent earth quack events. Although in most cases the damage was due to large movements of the Tower foundations, it seems relevant to determine the level of stresses the structures are subjected to during earth quacks.

Figure 2.1: typical lattice towers in Ethiopia (Ethio-Telecom Portal 2004)

2.2 Dynamic Response of Self-Supporting Lattice Towers

2.2.1 Response to wind

Chiu and Taoka (1973) studied the dynamic response of self-supporting lattice towers under actual and simulated wind forces. A three-legged, 46 m tall self-supported lattice tower was instrumented to study its dynamic response to wind forces. The tower was then idealized as a space truss with masses lumped at the horizontal panel points. The analysis of the field measurements indicated that the measured dynamic properties of the tower agreed with the calculated values. It was also found that the assumption of uncoupled motion in the two principal horizontal directions is valid. The study showed that for free-standing structures the fundamental mode of vibration is predominant. The average damping for the fundamental period was found to be 0.5% of the critical viscous damping value. Venkateswarlu et al.

(1994) studied the response of microwave lattice towers to random wind loads. The dynamic response was predicted using a stochastic approach. A spectral approach was proposed for calculating the along-wind response and the gust response factor. The gust response factor is defined as the ratio of the maximum expected wind load effect in a specified time period to the corresponding mean value in the same time period. A free-standing four-legged tower of 101 m height was used in this study. The variation of the gust response factor along the height of the tower was calculated with and without the contribution of the second and higher modes. It was found that the maximum contribution of the higher modes of vibration to the gust response factor is only of 2%, the gust response factor obtained using the proposed stochastic method varied between 1.547 and 1.584 along the height.

Using the formulae recommended by the Indian (IS 875-1987), Australian (AS 1170-2-1989), British (BS8100-1986) and American (ASCE 7-88-1990) standards, the values of the gust response factor were found to be 2.03, 2.21, 1.93 and 1.89 respectively. Comparing these results, it was concluded that the standards values were 20% to 40% higher than the values obtained using the spectral method. In a series of papers on the along-wind response of lattice towers, Holmes (1994, 1996) evaluated expressions for the gust response factor in a closed form for both the shearing force and the bending moment along the height of the tower. The tower used in this study was idealized with linear taper and a uniform solidity ratio so that the drag coefficient was kept constant. The reader may refer to Holmes (1994, 1995) for a complete explanation of the terms used.These expressions were then compared to the expressions currently used and both were found in agreement.The advantage of the proposed over the currently used expressions is the inclusion of more factors to account for the effects of various parameters associated with both the wind and the structure. An expression for the aerodynamic damping of the tower, due to the relative motion between the tower and the wind, as a ratio of the critical damping was also derived. In addition, a closed-form expression for the deflection at the top of the tower was derived considering three components of deflections namely the mean, background and resonant components. A study was then performed to investigate the effects of the height, taper ratio, and mean velocity on the gust response factors for shear force and bending moments. Finally the work was extended to predict an effective static load distribution, including the mean, back ground fluctuating and resonant components of the wind.

2.3 Seismic response

One of the first studies discussing earthquake effects on antenna lattice towers was presented by Konno and Kimura (1973). The study aimed at collecting information on tower mode shapes, natural frequencies and damping properties. One of the towers used in this study was instrumented when the 1968 Off-Tokachi earthquake occurred and simulation used a stick model of the tower with lumped masses and a viscous damping ratio of 1 %. In some members the forces due to earthquake loading were found to exceed those due to wind. Infact, some local damage and permanent deformations were observed at the base of the tower after the earthquake more recently, Mikus (1994) studied the seismic response of self- supporting telecommunication towers. The aim of this study was to improve the understanding of the response of these towers to earthquakes. Six towers with height from 20m to 90m were used in the study: bare towers only, no antennas, attachments, ancillary components etc. Three earthquake records were selected as the base excitation.

A detailed dynamic analysis was performed using modal superposition, and it was concluded that the use of the lowest four modes of vibration provided sufficient accuracy. The frequency of the first axial mode of the towers was found to be in the range of 11to43Hz.This range was either not included in the frequency content of the earthquake records used or corresponded to small amplitudes of input accelerations. It was therefore concluded that the vertical component of the earthquakes has negligible effect on the towers studied.

A first attempt to find an equivalent static method for the analysis of latticed self-supporting telecommunication towers was made by Gaivez (1995). The method was based on modal superposition taking the effect of the lowest three flexural modes of vibration into consideration. As self-supporting towers behave essentially as cantilever beams, Gaivez suggested the use of natural frequencies and mode shapes expressions developed for prismatic cantilevers. The effects of taper ratio and shear deformations were included by means of correction factors to the classical solution for prismatic Euler cantilevers.

The base excitation was assumed to be a sinusoidal wave with the maximum amplitude Ug being equal to the peak ground acceleration defined by the National Building Code of Canada (NBCC 1995) for the tower site.

The input excitation was assumed to be in resonance with each of the lowest flexural modes considered. Using the SRSS method for combining the relative modal accelerations, an acceleration profile along the height of the towers was defined. Detailed dynamic analysis using a total of 45 base accelero- grams was used to validate the method using three different towers. Based on these results, simplified acceleration profiles were proposed depending on the A/V ratio (peak ground acceleration to velocity ratio) of the accelero-grams. The inertia force was simply found by multiplying the acceleration profile by the mass profile. The structure was then analyzed under the effect of these equivalent "static'" inertia forces. Although simple, the method did not always give good estimates for the internal forces. For the main legs in general, the method gave conservative values accurate enough for preliminary design. Results were not systematically accurate and conservative for other members. The margin of error for the force prediction in the horizontal bracing was between -68% and +43%, and for cross bracing it was in the range of -35% to +23%,The method was limited to the tower geometry used in the study, i.e. having a taper ratio less than 14.5, and a total length to tapered length ratio less than 1.15,

2.3 Dynamic Response of Transmission Line Structures

The study of the dynamic problem arising from the unique case of the tower-conductor coupled system attracted several researchers. Some researchers investigated the dynamic loads on transmission towers due to galloping of the conductors (Baenzigeretal. 1994), conductor breakage (McClure and Tinawi 1987), ice shedding from the cables (Jamaleddine et al. 1993) and the free vibration of the coupled system (Ozonoet al.1988). However, most of the work done for the seismic analysis of the transmission lines involved the tower alone without considering the coupled tower-conductor problem. A review of the dynamic problem of the coupled tower-conductor system is indicated before summarizing the work done in seismic analysis of the transmission line systems. Ozono and Maeda (1993) studied the in-plane dynamic interaction between the tower and the conductors. The tower-conductor system was simplified assuming the tower to be a single lumped mass cantilever. Two models were used, the first with two spans of conductors and the second with only one span. For the two models, the conductors' ends not attached to the suspension tower were fixed to a rigid wall. The contribution of the natural modes of the conductors to the tension force exerted on the tower was obtained and it was found that at lower frequencies (less than 2 Hz) the contribution of the transverse wave modes to the tension force exerted on the tower is dominant. The results of this investigation suggested that the conductors play two important roles in the coupled system. The first effect is that when the conductors vibrate locally at a dominant frequency, their deformation induces a dynamic tension force on the tower. The second effect is that when the tower vibrates at a dominant frequency the conductors act as massless linking springs in the coupled system, i.e. their inertia effects are not significant. In an earlier study, Long (1974) investigated the effect of seismic excitation on a transmission tower neglecting the effects of the overhead conductors. The study was extended to evaluate the forces exerted by the conductors on the tower. The steel transmission tower was divided into two parts. The top part consisted of the prismatic part and the cross arms, and was treated as a flexible with uniform stiffness and mass, and treated as a uniform cantilever. The bottom part was assumed to be a rigid lumped mass. The total displacement of the flexible cantilever was then given by the following equation:

Kotsubo et al. (1985) performed dynamic tests on three transmission towers before and after installation of the conductors. The purpose of the study was to determine the effects of the conductors on the dynamic characteristics of the towers. The earthquake response of the towers was then evaluated numerically. The three towers used were two strain towers (with conductors directly anchored to the tower) with heights of 92.5m and68.5m, and a suspension tower with height of 92.2m. The results were reported for the case of the suspension tower only. The modes of vibration of the tower were calculated using both a plane truss model and a space truss model. Ambient vibration measurements for the tower were carried out before the installation of the cables. From these measurements and using power spectra, the natural frequencies, modes of vibration and damping properties were obtained. After the installation of the cables, vibration tests using an exciter were carried out. The exciter was set up on the third arm from the top the tower. It was observed that there were no significant changes in the natural frequencies and the modes of vibration of the tower before and after the cable stringing, which suggested that the dynamic interaction between the cables and towers is insignificant for suspension towers. The damping ratio of the tower was found to be in the range of 0.2 to2.0% of the critical viscous damping. The earthquake responses were then calculated using the plane truss model and the space truss model ignoring the presence of the cables. For the plane truss model, the responses were calculated for both the longitudinal land the transverse direction to the transmission line. It was concluded that it is sufficient to model the tower as a plane truss.

In a more recent study conducted by Li et al. (1991) mechanical models for long-span transmission line systems under earthquake effects were presented. This study included the derivation of mass and stiffness matrices for the tower-cable coupled system for the longitudinal and transverse directions. For the vertical direction the mass of the conductors was calculated and then lumped at the appropriate joints. For each of the three main directions a dynamic analysis was carried out using three earthquake records namely Qian'an (China), El Centro (USA) and Ninghe (China).The analyses were done for the following three cases for comparison:

I- The discretized model of the tower without the conductors.
II- Discretized model of the tower with the mass of the conductors lumped at relevant tower joints.
III- The coupled tower-conductor model.

It was found that for the vertical ground motion the seismic response of model II is greater than that of model I. For both the lateral and longitudinal ground motions, the response of model III was greater than that of model H, which in turn was greater than that of model I. It was concluded that the effects of the conductors on the seismic response of their supporting tower are not negligible and should be taken in to consideration.

2.4 Seismic Response of Tower-Shaped Structures

Due to the little amount of literature available on seismic analysis of latticed self-supporting towers, the search was directed towards other structures that behave essentially as cantilevers, namely offshore towers and intake-outlet towers. The aim of this research is to gain insight of the approaches used in analyzing such structures under seismic excitations and to find if a simplified method for analysis is available.

Penzien and Kaul (1972) studied the response of offshore towers to strong motion earthquakes. In their work, the response spectrum method of analysis was used and compared with the proposed stochastic method. In this proposed method, a mean ergodic Gaussian process of finite duration was used as the stochastic model for the horizontal ground acceleration. The aim of the study was to determine the transverse shear distribution and the overturning moment along the height of the towers without investigating the individual member forces. The towers were modeled as stick models with seven joints along the height on which the mass of the tower was lumped. A condensed stiffness matrix corresponding to the lateral displacements of the model was evaluated, and from the mass and stiffness matrices of the model, the Eigen properties of the towers (frequencies and mode shapes) were predicted. The distributions of the transverse shear and overturning moment were then calculated using the response spectrum of the earthquake excitation considering the contribution of the lowest three flexural modes.

The results were found to be comparable to those obtained with the more rigorous stochastic approach Anagonstopoulos (1982), in his work on modal solutions for the earthquake response of offshore towers, concluded that modal superposition gives good estimates of the overall response of the towers. For some members, however, the estimated value of the bending moment was in an errorofabout-60%, yet the difference in total stresses were less than 13% which can be reduced by increasing the number of modes in the summation. Due to the uncertainties in the earthquake loading, Anagonstopoulos suggested the use of more earthquake excitations instead of increasing the number of modes in the analysis. He also suggested that the inclusion of the lowest three modes in each of the three principal structural directions (the two horizontals and the vertical) would be adequate for design purposes. In the work reported by Chan (1987), response spectrum techniques for multi-component seismic analysis of offshore platforms were evaluated. Two plat forms were modeled taking into account the added mass of water. In this study three components of earthquake input were considered, two horizontal components with the ratio 0.67 : 1.0 and a vertical component with

0.5. The study aimed at evaluating the techniques used for modal combination as well as seismic component combination rules. The member forces and stresses calculated using different combination rules for both the modal summation and seismic components were compared with those obtained using detailed direct integration analysis. The different modal combination rules studied were the Square Root of Sum of Squares (SRSS), the Complete Quadratic Combination (CQC), and the American Petroleum Institute (API) method. For different directional seismic inputs, the SRSS and the Multi Component Quadratic Combination (MCQC) rules were used. It was concluded that all of these combination rules gave comparable results, and the CQC-SRSS rule was recommended because of its conservative results. As part of his study, Chan also checked the error resulting from neglecting the effect of higher modes (above the eleventh mode) in the analysis. He concluded that because all lower modes are horizontal, the vertical forces could be underestimated by a truncated analysis which in turn would affect the support design

2.5 Seismic response of intake outlet towers

Valliap--pan et al. (1980) investigated the effect of earthquakes on the in-take tower of Magrove Creek dam in Australia, using both dynamic and pseudo-static analyses. The design spectrum approach was used as a basis of the pseudo-static analysis considering only the lowest flexural mode of vibration. The modes have used was that report E5210 din Clough and Penzien (1993) in the form of a cosine function. The structure was then analyzed statically under the effect of inertia forces resulting from multiplying the acceleration profile due to the first mode shape by the mass .Detailed dynamic analysis was then performed and the results obtained for both analyses were compared.

From this comparison, it was concluded that the pseudo-static analysis is considering the lowest flexural mode is only an approximate solution. However, this conclusion might change if higher modes were included. A simplified method for seismic analysis of intake-outlet towers was developed by Chopra and Goyal (1991). The method was used to estimate the maximum forces in these towers using the design earthquake spectrum. A simplified step-by- step procedure based on the Stodola and Rayleigh methods for the calculation of the lowest two natural periods was suggested. It was demonstrated that considering the lowest two flexural modes of vibration is accurate enough for the preliminary design phase.

The maximum shear and bending moment at any section along the tower height were then found using the SRSS modal combination method. It is noted that the step-by-step method for estimating the lowest two natural periods is accurate if the variation in the tower cross- sectional properties can be expressed in a closed form. Since self-supporting lattice towers have usually irregular changes in their cross-sectional properties, the use of this method will only give very crude estimates for the natural periods. Also, a computer program was suggested for the implementation of the proposed procedure, which means that it is not such a "simplified" procedure.

2.6 Design Code Approaches for Seismic Analysis

Different design code approaches for the analysis of structures under earthquake loads need to be reviewed. Two types of structures are considered here, namely safety-related nuclear structures and buildings.

The seismic analysis of safety-related nuclear structures standard of ASCE (1986) suggests acceptable methods for the analysis and provides the methodology and the input ground motion to be used in calculating the response of such structures. This standard defines two methods for specifying the seismic input, namely response spectrum and input ground motion time history. The horizontal component of the response spectral ordinates (absolute acceleration Sa, spectral velocity Sv, and spectral displacement Sd) are obtained by applying dynamic amplification factors to the corresponding maximum values of ground motion (acceleration a, velocity v, and displacement d) obtained from the response spectrum. These Amplification factors depend on the amount of damping and are given as ratios of Sa/a, Sv/v, and Sd/d. The standard requires the use of two equal horizontal earthquake components. Two thirds of the horizontal component value is used as the vertical component of the input. If time histories are used, three different earth-quake records should be used in three orthogonal directions. These records must be selected so as to represent the site conditions. The standard recognizes four methods for the analysis of such structures: the direct integration method, the response spectrum method, the complex frequency method and the equivalent static method. The first three methods are well documented in text books (Bathe 1982, Gupta 1992, and Clough and Penzien 1993) and need not be reviewed here. As for the equivalent static method, the standard restricts its use to cantilever models with uniform mass distribution. Multi-degree of freedom models (MDOF) of cantilevers with non-uniform mass distribution can be analyzed using the static method if the maximum response is expected to result from loads in the same direction. In this case, the equivalent static load is determined by multiplying the structure's mass profile by a constant acceleration equal to 1.5 times the peak acceleration of the response spectrum. For cantilever structures with uniform mass, values of 1.0 and 1.1applied to the peak spectral acceleration are used to determine the tower base shear and base moment respectively. The justification of these values is not presented in the standard. The total response for the three components of seismic input is then obtained using the SRSS combination rule. Although the standard recommends this procedure for MDO models, the equivalent static method is limited to very simple models which have a dominant lowest frequency mode of vibration. The usual approach suggested in building codes (Paz 1994) for seismic analysis is to evaluate a global base shear value. The base shear is then distributed along the height of the structure assuming that the lowest mode of vibration is dominant and that the lateral displacement varies linearly. The National Building Code of Canada (NBCC 1995) specifies the minimum base shear for which the structure should be designed.

2.7 Codal -provisions in design of communication towers

The following are the steps involved in design of communication tower.

a. Selection of configuration of tower
b. Computation of loads acting on tower
c. Analysis of tower for above loads
d. Design of tower members according to codes of practices

Selection of configuration of a tower involves estabilishing of top width, bottom width, number of panels and their heights, type of bracing system and slope of tower.

2.8 Summary on the literature review

From this literature review it can be seen that seismic analysis of self-supporting telecommunication towers has received very little attention. The work done in other fields cannot directly be applied to self-supporting latticed towers. Since the designers are left without much guidance to assess if a detailed dynamic analysis is required , earthquake effects are usually ignored in the design office. For short towers and low risk seismic area this may be acceptable. However, in high risk seismic areas and for tall towers the designer should be able to perform at least a simple quasi-static analysis as a quick design check. Therefore, a simplified quasi-static method is proposed. The method is based on the modal superposition method and the response spectrum approach. It is anticipated that the proposed method will give reliable estimates of the member forces and in most cases performing a detailed dynamic analysis will become unnecessary. Based on the above studies the researcher of these study blevies that it is important to consider seismic loads on design of Telecom Tower in seismic zone of Ethiopia in addition to wind loads, that is why the researcher interested

CHAPTER THREE

3 DESIGN EXAMPLE For Wind Loads of Telecom Towers In EBCS EN 1991-1-4

3.1 Wind Load on Tower

The wind load on tower can be calculated using the EBCS EN 1991-1-4 .the designer should select the basic wind speed depending on the location of tower. The design wind speed is modified to induce the effect of risk factor (k_1), terrain coefficient (k_2) and local topography (k_3) to get the design wind speed V_z. ($V_z = k_1k_2k_3V_b$).

The design wind pressure P_z at any height above mean ground level is $0.6Vz^2$. The coefficient 0.6 in the above formula depends on a number of factors and mainly on the atmospheric pressure and air temperatures. Solidity ratio is defined as the ratio of effective area (projected area of all the individual elements) of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction. Force coefficient for lattice towers of square or equilateral triangle section with flat sided members for wind blowing against any face shall be as given EBCS EN 1991-1-4

The wind load acting on a tower can be computed as

$$\mathbf{F= C_{dt}A_eP_zk_2.}$$

For circular sections the force coefficient depends upon the way in which the wind flows around it and is dependent upon the velocity and kinematic viscosity of the wind and diameter of the section. The force coefficient is usually quoted against a non-dimensional parameter, called the Reynolds number, which takes account of the velocity and viscosity of the medium and the member diameter. The 50m tower located in Ethiopia is mounted with a hollow hemispherical dome of 2m diameter weighing 10kN.Compute the forces and stresses in members of various panels. The elevation of the tower is as shown below

3.2 Design of tower members

According to the EBCS EN 1991 the estimated tensile stresses on the net effective sectional areas in various members shall not exceed minimum guaranteed yield stress of the material. However in case the angle section is connected by one leg only, the estimated tensile stress on the net effective sectional area shall not exceed Fy, where Fy is the minimum guaranteed yield stress of the material. For structural steels confirming to the code the yield strength is 250 MPa. Generally Y_{st} 25 grade tubes confirming are used for tower members. As per the code estimated compressive stresses in various members shall not exceed the values given by the formulae in clause of the code. Accordingly the researcher tries to give emphasis as per the new code book of EBCS EN 1991 as follows.

3.3 Design Example

Data given:

Height of the tower = 50m

Base width= 6m

Top width= 2m

No. of panels = 20

Disk size = 2m diameter

Step 1: Wind force – From Basic wind speed = 22m/sec

Risk coefficient (k_1) = 1.06

$P_Z = 0.6\ V_Z\ = 0.6\ (39 \times 1.06 \times 1.2 \times k2)^2$

$=1476.6\ k_2^2\ N/m^2$

The values of $\mathbf{k_2}$ at different height is chosen from Table

Step 2: Basic assumptions:

1. Self-weight of the members are equally distributed to the two joints connected by the members

1. No load is applied at the middle of the k-braced joint but allocated to column joint

2. Dead and wind loads are increased by 15% for each joints to account for Gussets, bolts and nuts

3. Secondary members are assumed to be provided in the panel where batter starts (below the waist level in our case panels 16 to 20. So an additional load of 10% is accounted for in the case of provision of secondary members

4. The wind loads on the members are equally distributed to the connecting joints.

Step3: Calculation of solidity ratios:

3.1 Solidity Ratio

Solidity ratio (ϕ) = $\dfrac{\textbf{Projected area of all individual element}}{\textbf{Area enclosed by the boundary of the frame normal to the wind dirn}}$

Solidity ratios of panel 1 to 15 are calculated once as panels 1 to 15 are similar

$$\phi_{1\text{-}15} = \frac{15*2\,(2*0.15) + 15*2\,(21/2*2*0.05) + 16*2*0.045}{30*2} = 0.245$$

similarly for $\phi 16$

$$\phi_{1\text{-}16} = \frac{2*4.04*0.15+2*4.68*0.065+2.8*0.05}{2.4*4} = 0.204$$

$$\phi_{17} = \frac{2*4.04*0.5+2*5.14*0.065+1*3.6*0.065}{2.8*4} = 0.165$$

$$\phi_{18} = \frac{2*4.04*0.2+0.2*2*5.67*0.065+1*4.4*0.065}{4*4} = 0.165$$

$$\phi_{19} = \frac{2*4.04*0.2+2*4.479*0.065+1*5.2*0.065}{4.8*4} = 0.134$$

$$\phi_{20} = \frac{2*404*0.*2*5.016*0.065}{5.6*4} = 0.101$$

Step4 Calculation of bowl wind pressure

Bowl wind coeffs are $C_f = 1.4$ for wind from front

$C_f = 0.4$ for wind from rear

Wind pressure @ 50m above GL

Design wind pressure $\mathbf{P_z} = 1476.6(1.09)^2 = 1.754 kn/m^2$

Wind loads on dish are on front face $\mathbf{F_{dish1} = C_f * A_e * P_d}$

$\mathbf{F_{dish1}} = 1.4*\pi/4*22*1.754 = 7.71 kn$

On rear face

$\mathbf{F_{dish}} = 0.4*\pi/4*22*1.754 = 2.20 kn$

Step 5: The terrain factor (**k_2**), the solidity ratio and the design wind pressures @ various

Height are tabulated as shown – category 3 class B

Table 3.1: Terrain factor and solidarity ratio and design wind pressure in different heights.

Panel from top	Height in m from top	Terrain size,.k2, HT.coeff	Design wind pressure Pz=1476.6(k2)N/m2	Force coefficient Cf	Solidity ratio.	Pz*Cf N/m2
1 to 5	10	1.09,1.075,1.06	1706.4	3.075	0.245	5247.2
6 to10	20	1.06,1.045,1.03	1612.5	3.075	0.245	4958.4
11to15	30	1.03,1.005,0.98	1491.4	3.075	0.245	4586.1
16	34	0.98,0.964,0.948	1372.2	3.28	0.204	4500.8
17	38	0.948,0.926,0.904	1266.1	3.475	0.165	4399.7
18	42	0.904,0.88,0.856	1143.5	3.475	0.165	3975.7
19	46	0.856,0.832,0.808	1022.1	3.630	0.134	3710.2
20	50	0.808	964.0	3.795	0.101	3658.4

Step 6: Calculation of forces at different joints

The forces from the dish are transferred to two top most joints 1 and 4. The dish weight and wind force on the dish are equally distributed at the two joints

Panel 1 Leg: Length of the leg = 2m

Width of the leg = 0.15m

Since four Numbers of A 150 x 150 x 12 @ 0.272 kN/m

Self-weight of legs = 4 x 2 x 0.272 = 2.176 kN No. of legs exposed to wind = 2

Wind obstruction area = 2 x 2 x 0.15 = 0.6 m^2

wind load on leg = 0.6 x 5247.2= 3.148 KN

Diagonal Bracing ; Number of diagonal bracings = 8

Number of obstructing wind = 2

Size of diagonal bracing A50*50*6 @ 0.045 kn/m

Self-weight = 1/8*2*0.045 kn/m =1.018kn

Wind obstruction area 2*(2)1/2*2*0.05 = 0.283m^2

Wind load on diagonal bracing = 0.283*5247.2 = 1.485 kN

Horizontal Bracing (A 45*45*6)

No. of horizontal bracings = 8

No. of obstructing wind = 2

Self-weight of horizontal bracing = 8 x 2 x 0.04 = 0.64 kN

Wind obstruction area = 2 x 2 x 0.045 = 0.18m^2

Wind load on horizontal brac = 0.18 x 5247.2 = 0.945 kN

Total self-weight of leg, diag. brac and horizontal bracing

$\mathbf{F_V}$ = 2.176 + 1.018 + 0.64 = 3.834 kN

Total wind load on leg, diagonal and Hor. braces

$\mathbf{F_H}$ = 3.148 + 1.485 + 0.945 = 5.578 kN

These load are to be distributed to all the 8 joints connecting the elements (i.e. joints 1 to 8)

Load at each joint is increased by 15% to account for gussets, bolts and washers

F_{V1} vertical load on joints 1 to 8 = (1.15 x 3.834) / 8 = 0.551 kN

F_{H1} wind load on joints 1 to 8 = (1.15 x 5.576) / 8 = 0.802 kN

The self-weight of the dish is shared by joints 1 and 4

$F_{V\ DISH}$ = 10/2 kN = 5kN

Wind load on the dish is shared by joints **1, 2, 3** and **4**,

$F_{H\ DISH}$ = 7.71 / 4 = 1.93KN

Panel 2: Self weight of legs = 2.176 kN

Wind loads on legs = 3.148 kN

Self-weight of diag.Brace=1.018 kN

Wind load on diagonal.Brace =1.485 kN

Number of horizontal braces =4 and Number obstructing wind =4

Self weight of horizontal bracing = 4*2*0.04 = 0.32 kN

Wind obstruction area =1*2*0.045=0.09m^2

Wind load on horizontal bracing = 0.09*5247.2=472.2N

Vertical load due to leg and diagonal brace carried by joint 5 to12= 1.15(2.176+1.018)/8 =0.46KN

Vertical load due to hor. bracing carried by joints 9,10,and12

=1.15*(0.32)/4= 0.092KN

Wind load carried by joints 5 to 12 =1.15(3.148+1.485)/8= 0.666KN

Wind load carried by joints 9,10,11 and 12=1.15*0.472/4=0.136KN

Computation of loads at different joints are made of panel to panel from 2 to panel 5 are

Tabulated bellow.

Panel 6: Self weight of legs = 4 x 2 x 0.272 = 2.176 kN

Wind load = 0.6 x 4958.4 = 2.975 kN

Self-weight of Diag. Bracing. = 1.018 kN Wind load = 0.283 x 4958.4 = 1.403 kN

Self-weight of hor. bracings = 0.32 kN. Wind load = 0.09 x 4958.4 = 0.446 kN

Vertical load carried by joints 21 to 28 = (2.176 + 1.018) 1.15 / 8 = 0.46 kN

Wind load carried by joints 21 to 28 = (2.975 + 1.403) 1.15 / 8 = 0.63 kN

Vertical load due to horizontal brace. Carried by joints 25, 26, 27 and 28

=1.15*(0.32/4) = 0.092KN

Wind load carried by joints 25, 26,27 and 28 = 1.15*(0.446/4) = 0.128KN

Computations of loads @ different joints were done from 6 to 10 and are tabulated.

Panel 11: Vertical load carried by joints 41 to 48 = 0.46 kN

Wind load on the legs = 0.6 x 4586.1= 2.75 kN

Wind load on the Diag. Brac. = 0.283 x 4586.1= 1.3 kN

Vertical load due to Hor. Brac carried by joints 45, 46, 47 and 48 = 0.092 kN

Wind load carried by joints 41 to 48 = 1.15 (2.75 + 1.3)/8 = 0.582 kN

Wind load carried by joints 45 to 48 due to Hor. Brac. = (0.09 x 4586.1)/4 =103.18KN

Computations of loads at different joints was done from panel 11 to 15 and are tabulated.

Panel 16: Leg: A 150 x 150 x 15 @ 0.336 KN/m Length of the leg (L) = 4.04m

Width of the leg (B) = 0.15m Self weight of legs = 4 x 4.04 x 0.336 = 5.43 KN

No. of legs exposed to wind = 2

Wind obstruction area = 2 x 4.04 x 0.15 = 1.212 $\mathbf{m^2}$

Wind load on leg = 1.212 x 4500.8 = 5.454 KN

Diag. Brac: A 65 x 65 x 5 @ 0.049 KN/m

No. of bracing = 8 No. of obstructing wind = 2

Self-weight of diagonal brac. = 8 x 4.68 x 0.049 = 1.835 kN

Wind obstruction area = 2 x 4.68 x 0.065 = 0.6084 m2

Wind load on Diag. Brac = 0.6084 x 4500.8 = 2.74 kN

Horizontal Bracing A 65*65*5 @0.045

No. of bracing = 4 and No. of obstructing wind = 1

Self-weight of Hor. brac. = 4 x 2.8 x 0.045 = 0.504 kN

Wind obstruction area = 1 x 2.8 x 0.050 = 0.14 kN

Wind load on Hor. Brac = 0.14 x 4500.8 = 0.63 kN

Secondary bracings are accounted for so DL and WL is increased by 10% Vertical load carried by joints 61 to 68 = (1.25 / 5.43 + 1.835)/8 = 1.135 kN

Vertical load carried by joints 65 to 68 due to Hor. Brac. = 1.25 (0.504)/4 = 0.158 kN

Wind load carried by joints 61 to 68 = 1.25 (5.454 + 2.74)/8 = 1.28 kN

Wind load carried by joints 65 to 68 due to Hor. Brac = 1.25 (0.63) / 4 = 0.197 kN

Panel 17: Leg: A 150 x 150 x 16 @ 0.336 kN/m

Self-weight of legs = 4 x 4.04 x 0.336 = 5.43 kN

Wind obstruction area = 2 x 4.04 x 0.15 = 1.212 m2

Wind load on leg = 1.212 x 4399.7 = 5.332 kN

Diagonal Bracing; A 65*65*5@ 0.049kn/m

Self weight of diagonal brac. = 8 x 5.14 x 0.049 = 2.015 kN

Wind obstruction area = 2 x 5.14 x 0.065 = 0.6682 m2

Wind load on Diag. Brac = 0.6682 x 4399.7= 2.94 kN

Horizontal Brac: A 65 x 65 x 6 @ 0.058 kN/m Self weight of Hor. brac.

= 4 x 3.6 x 0.058 = 0.835 kN

Wind obstruction area = 1 x 3.6 x 0.065 = 0.234 m^2

Wind load on Hor. Brac = 0.234 x 4399.7 = 1.03 kN

Secondary bracings should be accounted for in this panel Vertical load carried by joints 69 to 72 = 1.25

(5.43 + 2.015)/8 = 1.163 kN

Vertical load carried by (Due to horizontal brac.) joints 69 to 72 = 1.25 (0.835)/4 = 0.261 kN

Wind load carried by joints 65 to 72 = 1.25 (5.332 + 2.94)/8 = 1.29 kN

Wind load carried by joints 69 to 72 due to Hor. Brac = 1.25 (1.03) / 4 = 0.332 kN

Panel 18 : Leg: A 200 x 200 x 15 @ 0.454 kN/m

Self-weight of legs = 4 x 4.04 x 0.454

Wind obstruction area = 2 x 4.04 x 0.2 = 1.616 m^2

Wind load on leg = 1.616 x 3973.7 = 6.42 kN

Diag. Brac: A 65 x 65 x 6 @ 0.058 kN/m

Self-weight of diagonal brac. = 8 x 5.67 x 0.058 = 2.63 kN

Wind load on Diag. Brac = 2 x 5.67 x 0.065 x 3973.7 = 2.93 kN

Horizontal Brac: A 65 x 65 x 6 @ 0.058 kN/m

Self weight of Hor. brac. = 4 x 4.4 x 0.058 = 1.02 kN

Wind load on Hor. Brac = 1 x 4.4 x 0.065 x 3973.7 = 1.14 kN

Vertical load carried by joints 69 to 79 except 74, 76, 78, 80

= 1.25 (7.34 + 2.68)/8 1.56 kN

Vertical load carried by joints 73, 75, 77, 79 (Due to horizontal brac.) = 1.25 (1.02)/4 = 0.32 kN

Wind load carried by joints 65 to 79 except 74, 76, 78, 80 = 1.25 (6.42 + 2.93)/8

= 1.46 kN

Wind load carried by joints 73, 75, 77, 79 due to Hor. Br. =1.25 (1.14) / 4= 0.356 kN

Panel 19: Leg: A 200 x 200 x 15 @ 0.454 kN/m Self weight of legs = 4 x 4.04 x0.454

= 7.34 kN

Wind load on leg = 2 x 4.04 x 0.2 x 3710.2 = 6 kN

Diag. Brac: A 65 x 65 x 6 @ 0.058 kN/m

Self-weight of diagonal brace. = 8 x 4.79 x 0.058 = 2.22 kN

Wind load on Diag. Brac = 2 x 4.79 x 0.065 x 3710.2 = 2.31 kN

Horizontal Brac: A 65 x 65 x 6 @ 0.058 kN/m

Self-weight of Hor. brac. = 4 x 5.2 x 0.058 = 1.21 kN

Wind load on Hor. Brac = 1 x 5.2 x 0.065 x 3710.2 = 1.254 kN

Vertical load carried by joints 73 to 88 except 74, 76, 78, 80, 82, 84, 86, 88 =

1.25 (7.34 + 2.22)/8 = 1.494 kN

Vertical load carried by joints 81, 83, 85, 87 (Due to horizontal brac.) = 1.25 (1.21)/4

= 0.378 kN

Wind load carried by joints 73, 75, 77, 79, 81, 83, 85, 87 = 1.25 (6 + 2.31)/8

= 1.3 kN

Wind load carried by joints 81, 83, 85, 87 due to Hor. Brac = 1.25 (1.254) / 4

= 0.392kN

Panel 20: Leg: A 200 x 200 x 15 @ 0.454 kN/m Self weight of leg = 4 x 4.04 x 0.454

= 7.34 kN

Wind load on leg = 2 x 4.04 x 0.2 x 3658.4 = 5.91 kN

Diag. Brac: A 65 x 65 x 6 @0.058 kN/m

Self-weight of diagonal brac. = 8 x 5.02 x 0.058 = 2.33 kN

Wind load on Diag. Brac = 2 x 5.02 x 0.065 x 3658.4 = 2.39 kN

Vertical load carried by joints 81, 83, 85, 87, 89, 90, 91, 92 = 1.25 (7.34 +2.33)/8 = 1.51 kN

Wind load carried by joints 81, 83, 85, 87, 89, 90, 91, 92 = 1.25 (5.91 +2.39)/8 = 1.3 kN

Computation of loads at different joints is made panel by panel and the nodal loads are superposed and tabulated in the following sections. The tower is symmetrically loaded in the XY plane and so nodal loads are tabulated for joints which are in the front plane.

Calculation of forces in the members By symmetry the two planes are identical the front plane is analysed and forces are resolved. The tower is analysed for three basic static loads

- Self weight of the tower
- Superimposed load from Hemispherical Dome
- Wind Loads
  - ❖ Acting diagonal to the tower
  - ❖ Acting parallel to face

Table 3.2: summary of joint forces

Joint No	Self.wt.(kN)	Wind load (kN)	Joint No	Self WT (kN)	Wind load(kn)
1	5 + 0.551 = 5.551	0.802+1.93 =2.732	2	0.551	0.82+1.93 =2.732
5	0.551+0.46+1.011 =6.56	0.802+0.666 =1.468	6	0.551+0.46+1.011 =1.562	0.802+0.66 =1.468
9	0.46+0.092+0.46+1.012 =7.574	0.666+0.136+ 0.666=1.468	10	0.46+0.092+0.46+ 1.012=2.574	0.666+0.136+0.66 =1.468
13	0.46+0.092+0.46+1.012	0.666+0.136+	14	0.46+0.092+0.46+	0.666+0.136+0.666

	=8.586	0.666=1.468		1.012=3.586	=1.468
17	0.46+0.092+0.46+1.012 =9.598	0.666+0.136+ 0.666=1.468	18	0.46+0.092+0.46+ 1.012=4.598	0.666+0.136+0.666 =1.468
21	0.46+0.092+0.46+1.012 =10.61	0.666+0.136+ 0.63=1.432	22	0.46+0.092+0.46+ 1.012=5.61	0.666+0.136+0.63 =1.432
25	0.46+0.092+0.46+1.012 =11.622	0.63+0.128+ 0.63=1.388	26	0.46+0.092+0.46+ 1.012=6.622	0.63+0.128+0.63 =1.388
29	0.46+0.092+0.46+1.01 =12.634	0.63+0.128+ 0.63=1.388	30	0.46+0.092+0.46+ 1.012=7.634	0.63+0.128+0.63 =1.388
33	0.46+0.092+0.46+1.012 =13.646	0.63+0.128+ 0.63=1.388	34	0.46+0.092+0.46+ 1.012=8.646	=0.63+0.128+0.63 =1.388
37	0.46+0.092+0.46+1.012 14.658	0.63+0.128+ 0.63=1.388	38	0.46+0.092+0.46+ 1.012=9.658	0.63+0.128+0.63 =1.388
41	0.46+0.092+046+1.012	0.63+0.128+	42	0.46+0.092+0.46+	0.63+0.128+0.63

	=15.67	0.63=1.34		1.012=10.67	=1.34
45	0.45+0.092+0.46+1.012 =16.682	0.582+0.103+ 0.582=1.267	46	0.46+0.092+0.46+ 1.012=11.682	0.582+0.103+0.582 =11.682
49	0.46+0.092+0.46+1.012 =17.694	0.582+0.103+ 0.582=1.267	50	0.46+0.092+0.46+ 1.012=12.694	0.582+0.103+0.582 =1.267
53	0.46+0.092+0.46+1.012 =18.706	0.582+0.103+ 0.582=1.267	54	0.46+0.092+0.46+ 1.012=13.706	0.582+0.103+0.582 =1.267
57	0.46+0.092+0.46+1.012 =19.718	0.582+0.103+ 0.582=1.267	58	0.46+0.092+0.46+ 1.012=14.718	0.582+0.103+0.582 =1.267
61	0.46+0.092+1.135+1.687 =21.405	0.582+0.103+ 1.28=1.965	62	0.46+0.092+0.46+ 1.012=16.405	0.582+0.103+1.28 =1.965
65	1.135+0.158+1.163+2.45 =23.861	1.28+0.197+ 1.29=2.767	66	0.46+0.092+0.46+ 1.012=18.861	1.28+0.197+1.29 =2.767
69	1.163+0.261+1.56+2.984 =26.845	1.29+0.322+ 1.46=3.072	70	1.135+0.158+ =21.845	1.29+0.322+146 =3.072

73	1.56+0.32+1.494+3.374 =30.219	1.46+0.356+ 1.3=3.116	75	1.56+0.32+1.494 3.374=25.219	1.46+0.356+1.3 =3.116
81	1.494+0.378+1.51+3.382 =33.601	1.3+0.392+1.3 =2.99	83	1.494+0.378+1.51 3.382=28601	1.3+0.392+1.3 =2.99
89	33.601+1.51 =35.11	1.3	90	1.51 =30.111	1.3

Panel 15:

1. considering self-weight of the tower

The leg A 150 x 150 x 12 will be maximum stressed in this panel. So this panel is chosen. The self-weight acting on joints 61 and 62 is taken .The leeward leg 2 will be in compression and also the windward leg 1

F1 = F2 = 16.405 kN (compression)

1. **Considering superimposed load from hemispherical dome**:

The front plane takes half the self-weight = 5kN.The self-weight of the dome will create a moment with respect to center of planar truss. The eccentric load of 5 kN is transferred as a concentric load of 5 kN acting at the center of planar truss and an anticlockwise moment of 7.5 kN.m as shown. Due to self-weight both the legs F1 and F2 will be in compression

F1 = F2 = 2.5 kN (compression)

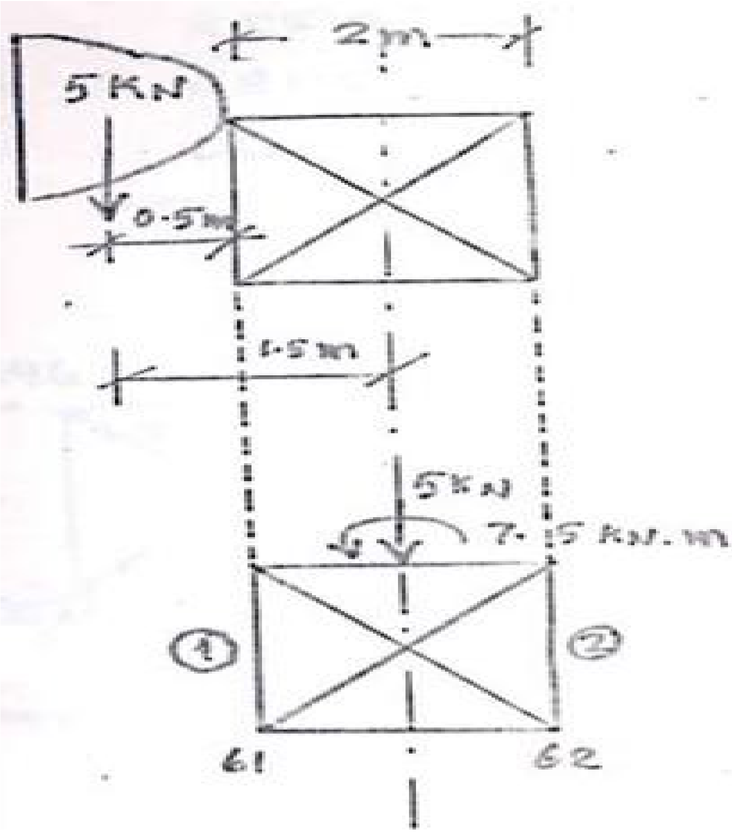


Figure 3.2 summery of loads from hemispherical dome

The moment will cause compression on the windward side and tension on the leeward side.

F1 = 7.5 / 2 = 3.75 kN (compression)

F2 = 7.5 / 2 = 3.75 kN (tension)

Net force on F1 = 3.75 + 2.5 = 6.25 kN (compression) Net force on F2 = -3.75 + 2.5 = 1.25 kN (tension)

The moment due to dome and self weight are carried entirely by legs.

3 .Considering wind load condition

(i) Wind parallel to the face of the frame

The sum of the wind forces up to panel 15 and also the bending moment due to wind load about point 0 (the point of intersection of Diag. Brac.) is taken

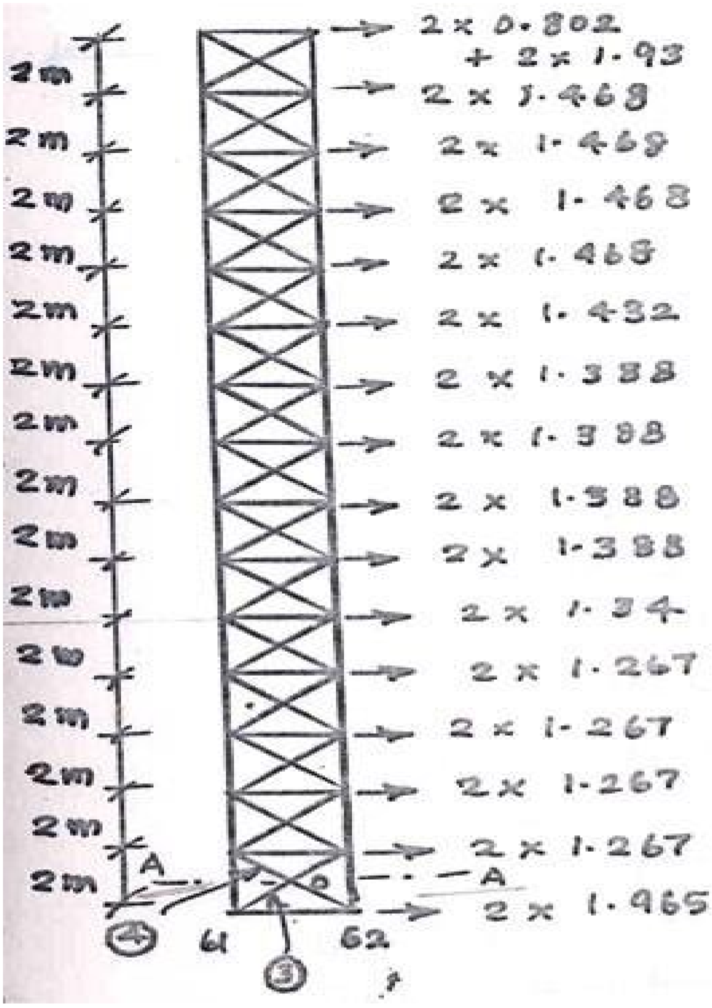


Figure 3.3; wind parallel to the face of the frame

Total wind load above the level 'AA'

FLAT1 = 2 x 0.802 + 2 x 1.93 + 4 x 2 x 1.468 + 2 x 1.432 + 4 x 2 x 1.388 + 2 x

1.34 + 4 x 2 x 1.267

FLAT1 = 43.992 kN

Moment due to wind

MW1 = (1.604 + 3.86) x 29 + 2.936 x 27 + 2.936 x 25 + 2.936 x 23 + 2.936 x

21 + 2.864 x 19 + 2.776 (17 + 15 + 13 + 11) + 2.68 x 9 + 2.534 (7 + 5 + 3 + 1)

MW1 = 714.85 kN.m

This external wind moment has to be resisted by internal couple. This moment will cause tension of the windward leg and compression on the leeward leg

F1 = MW1 / 2 = 714.85 / 2 = 357.43 kN (tension)

F1 = 357.43 kN (tension) F2 = 357.43 kN (compression)

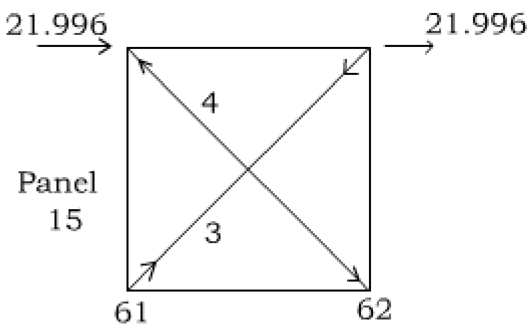


Figure 3.4; Summary of joints moments

The lateral force of 43.992 KN is shared by the diagonal bracings equally and the tension diagonal is considered as effective taking moment about joint 62

$43.992 = F3\sqrt{2}$

F3 = 31.11 kN tension

F4 = 31.11 kN compression

(ii) Wind wards acting along diagonal:

when the wind is parallel to the diagonal, the wind pressure coefficient is taken 1.2 times that of parallel to the plane ,However the wind pressure on the dish is reduced as the wind is at 45^o to the front of the dish. Wind pressure on the dish = 2 x 3.86 x Sin 45^o =5.46

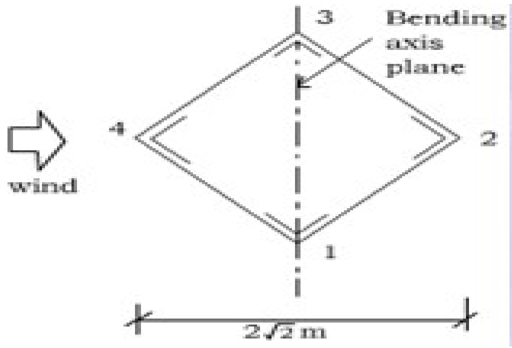


Figure 3.5; Wind acting along diagonal

Considering the tower as a space frame:

The wind load on the four joints together can be obtained. By multiplying the loads by 1.2

So total horizontal load due to wind

FLAT 2 = 5.46 + 1.2 x 2 (43.992 - 3.86)

FLAT 2 =101.78 kN

Similarly the bending moment of all the wind forces along the diagonal about point 0

MW2 = 1.2 x 2 {714.85 - (3.86 x 29)} + 5.46 x 29

MW2 = 1605.32 kN.m. Since the legs are upright, the horizontal force is registered by the braces and the forces in the braces will be equal and opposite.

The forces have to be resolved in the horizontal plane and then parallel to the diagonal.

Let FD = force in each brace (tension or compression)

The total force from braces in the horizontal plane along the tower diagonal is

= 8 FD $\cos 45^o . \sin 45^o$ =4FD

Equilibrium in the horizontal direction gives

4 FD = 101.78 kN FD = 25.45 kN

This value is less than that of case 1. Therefore the forces in braces are controlled by the load condition wind parallel to the frame. The bending moment is resisted by the pair of extreme legs 2 and 4. Forces in legs 3 and 1 will be zero as they lie in the bending axis Ref. Fig.

F1 = F3 = 0

F2 = MW2 (2)1/2* 2= 1605.32 (2)1/2* 2

F2 = 567.57 kN (compression)

F4 = 567.57 kN (tension)

Maximum compressive force on the leg = 567.57 + 16.405 - 1.25 = 582.73 kN

Leg A 150 x 150 x 12 @ 0.272 kN / m A = 3459 mm2; rmin = 29.3 mm

Leff = 0.85 x 2000 = 1700mm; Leff / ry = 1700 / 29.3 = 58.02

sac from table 5.1 = 124 N/mm2 can be raised by 25%. Since wind is considered: sac = 1.25 x 124 = 155 N/mm2

Actual stress sc = (582.73 x 103) / 3459 = 168.5 N/mm2

Diag. Brac: The tension member is considered effective

Force in the bracing = 31.11 kN

Size A 50 x 50 x 6 mm A = 568 mm2

Check the adequacy of the section as a tension member

Panel 20: Leg: A 200 x 200 x 15 @ 0.454 kn/m

1. Self weight acting at the bottom most panels

F1 = F2 = 30.111 kn (compression)

The leg is checked at the mid height as buckling will occur midway between the nodes

. Considering superimposed load from hemispherical dome

Due to moment F1 = 7.5 / 5.6 = 1.34 kn (compression) F2 = 1.34 kN (tension)

Due to self weight F1 = 2.5 kN (compression)

F2 = 2.5 kN (compression)

Net forces F1 = 1.34 + 2.5 = 3.84 kN (compression) F2 = -1.34 + 2.5 = 1.16 kN (compression)

3. Considering wind load condition:

(a) Wind parallel to the face of the frame:

Total wind load above level 'BB'

FLAT 3 = 43.992 + 2 x 1.965 + 2 x 2.767 + 2 x 3.072 + 2 x 3.116 + 2 x 2.99

FLAT 3 = 71.812 kN

MW3 = (1.604 + 3.86) x 48 + 2.936 (46 + 44 + 42 + 40) + 2.864 x 38 +

2.776 (36 + 34 + 32 + 30) + 2.68 x 28 + 2.534 (26 + 24 + 22 + 20) + 3.93 x 18

+ 5.534 x 14 + 6.144 x 10 + 6.232 x 6 + 5.98 x 2 MW3 = 1809.704 kN.m

Force in the legs and braces

$F_1 = M_{W3} / a = 1809.704 / 5.6 = 323.16$ kN

$F_1 = 323.16$ kN (tension)

$F_2 = 323.16$ kN (compression)

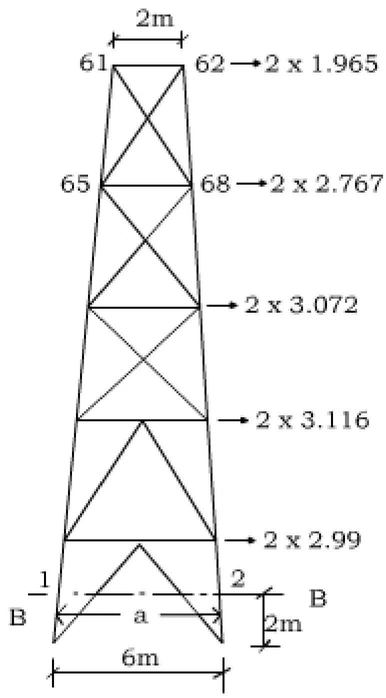


Figure 3.6; concentrated wind loads on the Towers

The lateral force of 71.812 kN is shared by the diagonal bracings equally and the tension diagonal is considered effective taking moment about joint 9^0

35.906 x 4 = F3 x 4.8

F3 = 29.92 kN (tension)

F4 = 29.92 kN (compression)

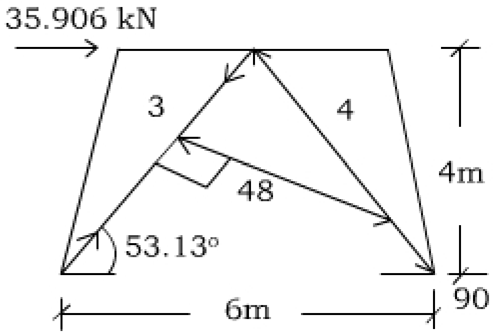


Figure 3.7 Wind acting parallel to the diagonal

Wind load is increased by 1.2 times that of parallel to the frame. P.47 code. However wind pressure on the dish is reduced as the wind is 45^{o} to the front of the dish

Wind pressure on dish = 5.46 kN

Considering the tower as a space frame the wind load on the four joints together can be obtained by multiplying the load by 1.2

So, total horizontal load due to wind FLAT 4 = 5.46 + 1.2 x 2 (71.812 - 3.86)

FLAT 4 = 168.55 kN

Similarly the bending moment of all the wind forces along section 'BB'

MW4 = 1.2 x 2 {1809.704 - (3.86 x 48)} + 5.46 x 48

MW4 = 4160.7 kN.m

The horizontal forces are resisted by the braces these forces have to be resolved in the horizontal plane and then parallel to the diagonal.

Let F_d be the force in each brace tension or compression. The total force is resisted by these 8 braces

4Fd cos 53.13^{o} (cos 37.47^{o} + cos 52.59^{o}) = 168.55 F_d = 50.12 kN (tension or compression)

This is more than the value with wind parallel to the frame. The bending moment MW4 is resisted by the pair of extreme legs which does not lie on the bending axis

F1 = F3 = 0

F2 = MW4 / a^2 = 4160.7 / 5.6*(2)1/2 =525.4KN

F2 =525.4KN (compression)

F4 = 525.4 KN (Tension)

Maximum compressive force will be on leg 2

= 30.111 + 1.16 + 525.4

F2 = 556.67 kN (compression)

Leg A 200 x 200 x 15 @ 0.454 kN/m

A = 5780 mm2; ry = 39.1 mm

Lef = 0.85 x 4040 = 3434mm

Lef / ry = 3434 / 39.1 = 87.83 Refer Table 5.1

δ_{ac} = 86 N / mm^2

Since wind is considered allowable stresses are raised by 25%. So δ_{ac} = 1.25 x 86 = 107.5 N / mm^2

Actual stress δ_c = 556.67 / 5780 = 96.31 N / mm^2

δ_{ac} and δ_c Safe

Chapter Four

4.1 C SHARP PROGRAMING LANGUAGE

C# (pronounced as see sharp) is a multi-paradigm programming language encompassing strong typing, imperative, declarative, functional, generic, object-oriented (class-based), and component-oriented programming disciplines. It was developed by Microsoft within its .NET initiative and later approved as a standard by Ecma (ECMA-334) and ISO (ISO/IEC 23270:2006). C# is one of the programming languages designed for the Common Language Infrastructure.

C# is a general-purpose, object-oriented programming language. Its development team is led by Anders Hejlsberg. The most recent version is C# 7.0 which was released in 2017 along with Visual Studio 2017.But due to inavilability and cost of the software the researcher is limited to work with 2013 version of the programing language.

4.2 Design goals of C#

The ECMA standard lists these design goals for C#

The language is intended to be a simple, modern, general-purpose, object-oriented programming language.

The language, and implementations thereof, should provide support for software engineering principles such as strong type checking, array bounds checking, detection of attempts to use uninitialized variables, and automatic garbage collection. Software robustness, durability, and programmer productivity are important.

The language is intended for use in developing software components suitable for deployment in distributed environments.

Portability is very important for source code and programmers, especially those already familiar with C and

C++ Support for internationalization is very important.

C# is intended to be suitable for writing applications for both hosted and embedded systems, ranging from the very large that use sophisticated operating systems, down to the very small having dedicated functions.

Although C# applications are intended to be economical with regard to memory and processing power requirements, the language was not intended to compete directly on performance and size with C or assembly language

4.3 Syntax

The core syntax of C# language is similar to that of other C-style languages such as C, C++ and Java. In particular: Semicolons are used to denote the end of a statement. Curly brackets are used to group statements. Statements are commonly grouped into methods (functions), methods into classes, and classes into namespaces.

Variables are assigned using an equals sign, but compared using two consecutive equals signs.

Square brackets are used with arrays, both to declare them and to get a value at a given index in one of them

Libraries

The C# specification details a minimum set of types and class libraries that the compiler expects to have available. In practice, C# is most often used with some implementation of the Common Language Infrastructure (CLI), which is standardized as ECMA-335 Common Language Infrastructure (CLI)

Having taken all the courses on Java and C++ programming language the researcher wrote the following programing syntax which can calculate the deflection of a tower using the applications of concentrated lateral load P. The length of the tower, the width of the tower, the depth @o and the depth @ L, number of division, load applied and modules of elasticity of the steel should be feed in their respective units by the user. Accordingly the researcher designs the following version of calculator. Notice that the writer only limited to the number of divisions (10,15,20,50 or 100)

Her is the beginning of the language. The entire part is attached on Appendex C and the reader can get the full syntax on the soft copies of the thesis

```
using System;
using System.Collections.Generic;
using System.ComponentModel;
using System.Data;
using System.Drawing;
using System.Linq;
using System.Text;
using System.Threading.Tasks;
using System.Windows.Forms;
using System.Windows.Forms.DataVisualization.Charting;
namespace deflection_and_rotation_calculater
{
    public partial class Form1 : Form
    {
        public Form1()
        {
            InitializeComponent();
        }
        private void Calculate_Click(object sender, EventArgs e)
        {
           // DEFINE VARIABLES
            Int32 length;
            Double b;
            Double o;
            Double l;
            Double x;
            Double n;
            Double p;
            Double E;
            Double deflectionAtx;
            Double h;
            Double s;
            Double n1;
            Double n2;
            Double n3;
            Double n4;
```

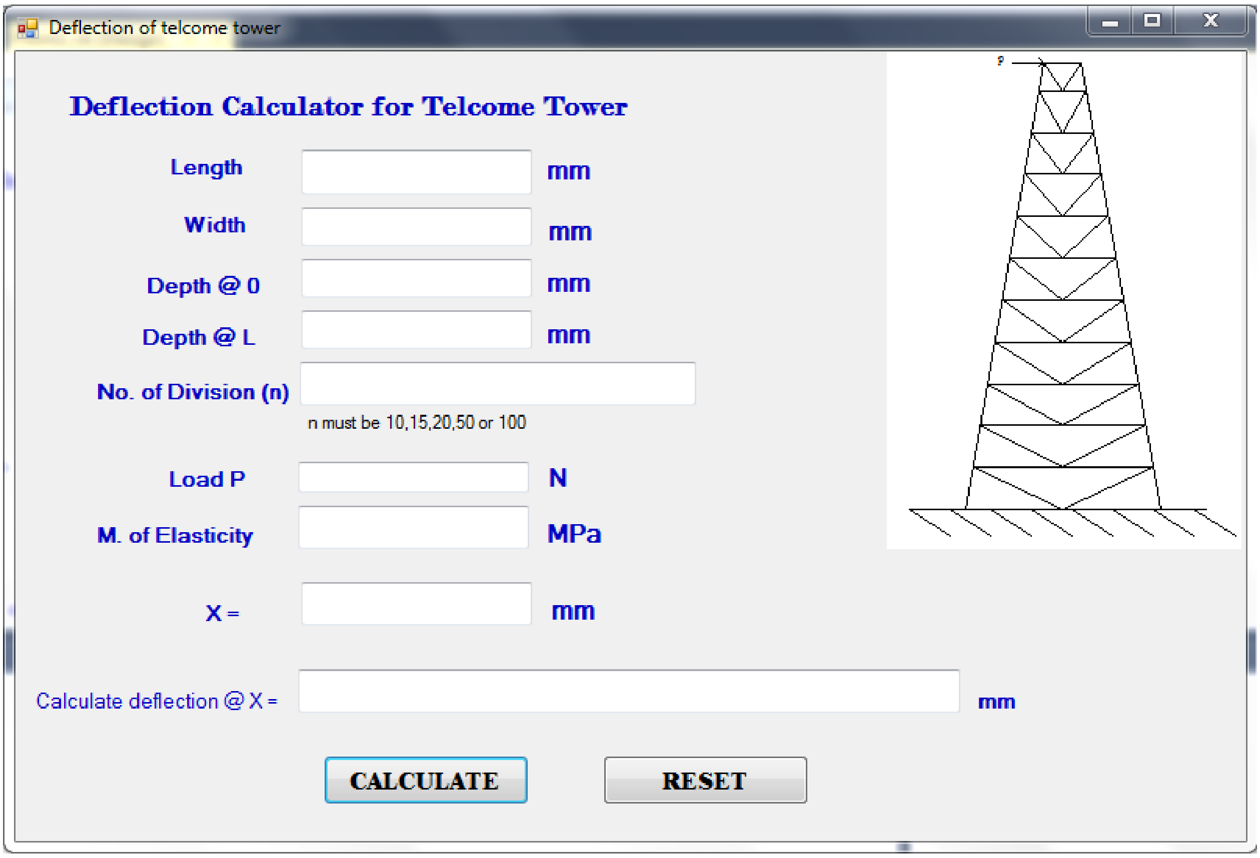
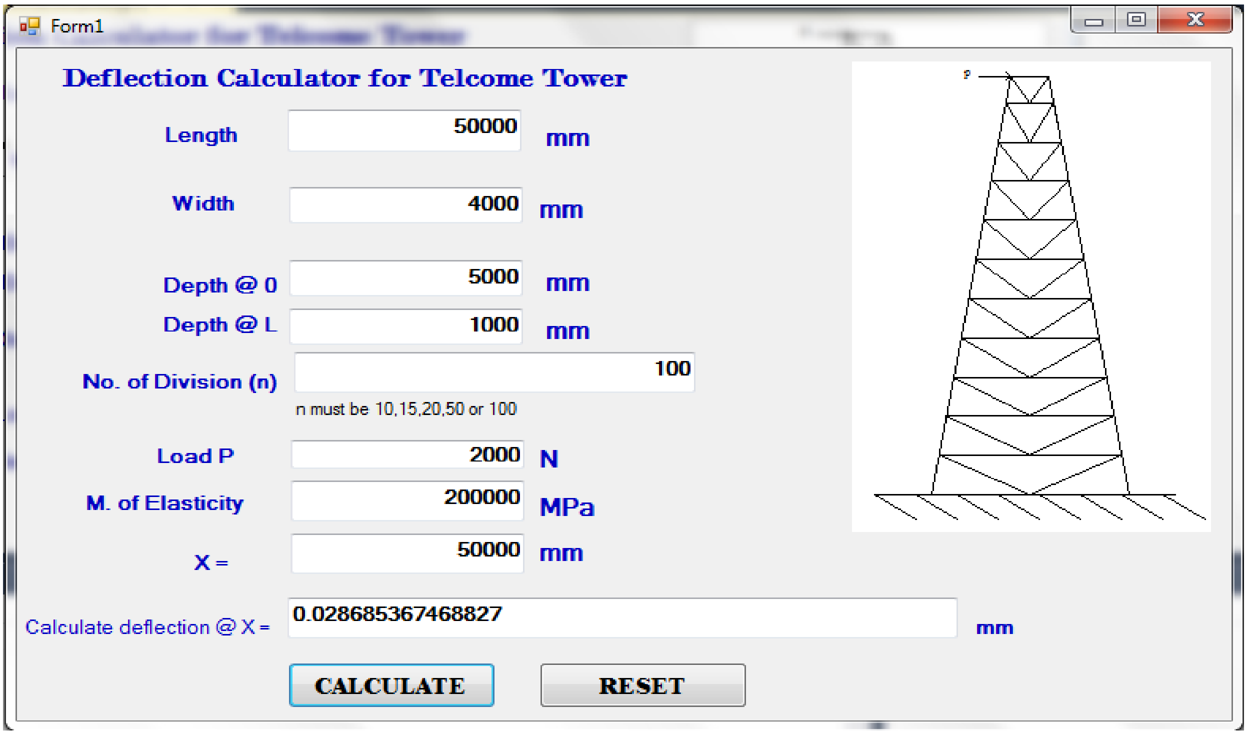


Figure 4.1; Deflection Calculator of the Tower (created by the researcher and generated from c# programming language)

4.4; Example



Assume the length of the Tower is 50m and the width @ the Top is 4m,Depth@ x=0 and @x=L is respectively 5m and 1m and number of division of the Tower is 100 strips and assume the consentrated lateral load is 200N and the modulus of elasticity of the structural steel is 200000MPA and if the concentrated load is applied @x=50m(top of the Tower) then the calculated horizontal drift or horizontal displacement of the Tower will be 0.028685367468827mm.Notice that these calculator can only calculates for number of divissions (10,15,20,50 and100). The reader can reset and pass to some other compitations simply by clicking reset. The reader can get little parts of these syntax at the Appendix but the whole part is presented and included on the soft copies of these thesis.

4.5 Compression of Results of C# Vs Sap2000 Model

Her the researcher tries to compare and contrast the horizontal drifts made by a typical Telecommunication Tower found her in Ethiopia due to horizontal load from the result obtained from C# calculator and Sap is presented below. For the convenience of comparison the researcher models a simple chevron V brased Tower on Sap2000 whose modules of elasticity is 200000MPA, length of the tower is 50m and all the parameters were taken in to consideration. The concentrated load P=200N applied @ the top of the tower. Accordingly the horizontal drift of the Tower is 0.0287982mm, but the one obtained from the C# calculator is 0.028685 which is quite similar with that of Sap. Notice that due to lack of finance to purchase the original version of the software the researcher is only limited to do with trial version of the software, so the small difference seen above arose from the originality of the software. Finally the researcher concludes that the developed calculator is almost perfect despite with its all limitations.

CHAPTER FIVE

5 MODELING AND LOADING OF STRUCTURAL SYSTEMS

5.1 Introduction

Telecommunication towers are complex structures with many members. The large number of members makes these towers difficult to analyze by hand, due to the many calculations necessary. To make analysis run more quickly and accurately, computer software has been designed to do finite element and modal analysis. These tools allow for a model to be created fairly quickly and for member sizes and connection types to be changed easily. They also allow for multiple loads and load combinations to be applied to the structure at the same time. For Analysis of the towers in Ethiopia modeling and analysis tools have been chosen. SAP2000 was chosen to compute wind and earth quack on the tower. It follows the TIA222 code and has all past versions of the code programmed into the software. Since there is no available software which can model both wind and earth quack effects together the researcher is limited to work with SAP2000

.

5.2 SAP2000 v9.0

SAP 2000 is a static and dynamic structural analysis program that includes linear and non-linear analysis capabilities. Seismic analysis can be performed using SAP 2000, and the ground motion can be modeled using spectrum or time history functions. Of particular interest for this thesis were the dynamic modeling capabilities of SAP 2000, the researcher is only titled to performed using response spectrum analysis, and combinations of loading scenarios. Modal analysis was performed using Eigenvector analysis for response spectrum function and Ritz vector for time history function.

SAP 2000 allows the user to input the response spectrum function. Preprocessing in SAP2000 utilizes a graphical interface for defining the tower geometry and properties of members and for defining loads and load combinations. Post processing provides output for internal forces and moments, displacements, mode shapes, and design checks.

SAP 2000 has a fairly complicated user interface, however it allows for more complicated models and analyses of structures. To begin in SAP 2000, the user is required to create a coordinate or grid system, either rectangular or cylindrical coordinates, in which the model of the structure will be drawn. The frame work of the model is drawn by defining the grid. Members are then drawn from intersections of the gridlines to create the basic model. Once the simple model had been drawn, member sizes and joint releases can be defined. By picking a member, a section can be assigned by selecting the section from a drag down menu. Following the defining of members and sections, loads and loading combinations need to be defined. Loads can be defined by using one of the pull down menus at the top of the user interface. Selecting a point or member, loads can be assigned similar to the way sections are defined. Along with simple point and distributed loads, loading functions can be defined also. Important functions that can be loaded on the tower using SAP2000 are time history functions and response spectrum functions. These functions can be imported into SAP and then a modal analysis of the tower can be executed. This is especially important for the research performed on the Telecommunication tower, as these two types of functions are how earthquake loading can be modeled but the researcher is limited to response spectrum function.

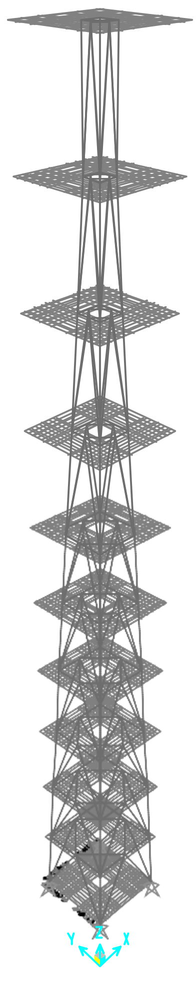


Figure 5.1 Finite element model

5.3 Analysis of wind loading

5.3.1 Introduction

The major environmental factors affecting communication towers in Ethiopia is wind load. The Ejere City tower is analyzed for wind loading conditions using SAP2000. The program is a compilation of spreadsheets that aid in the modeling of geometry of the tower, and application of external loads such as antennas, dishes and feed lines. SAP2000 analyzes the towers using the TIA-222-F standard or any of the previous versions of the TIA/EIA standards. For steel analysis, the program uses the AISC ASD 9th edition. Linear and nonlinear (p-delta) analysis can be performed to determine the displacements and forces in the structure. Once analysis has been performed, SAP2000 creates an extensive report consisting of all inputs into the software and results for the tower. The results include stresses in each member of the tower and whether or not the members fail or pass with respect to the standards and codes that were applied. The sap2000 report to these study is compiled on Appends B

5.3.2 Ejere 02 Tower Model Details

The Tower is located in southern part of Ethiopia in zone 4 of the seismic map. This area is seismically active; therefore a seismic analysis is also needed to determine the stability of the tower. The seismic analysis was done using SAP2000 v9.0.

SAP2000 is a static and dynamic structural analysis program that includes linear and non-linear analysis capabilities. Seismic analysis can be performed using SAP 2000 and the ground motion can be modeled using response spectrum. Of particular interest for this thesis were the dynamic modeling capabilities, which can be performed using response spectrum analysis and combinations of loading scenarios. Modal analysis was performed using Eigenvector analysis for response spectrum. SAP2000 allows the user to input the response spectrum function. Preprocessing in SAP 2000 utilizes a graphical interface for defining the tower geometry and properties of members and for defining loads and load combinations. Post processing provides output for internal forces and moments, displacements, mode shapes, and design checks.

The tower was modeled using the information provided by structural tower drawings (Ethio- telecom tower design detail manual) the reader can see in Appedex A . Leg, diagonal bracing, horizontal bracing sizes and connection b/n members were used as provided drawings. Selected element sizes were verified by field measurements. Tower attachments as shown in the drawings were included in the model. Design drawings were modeled by SAP2000 to predict the response under earthquake loads. The tower was modeled as a frame structure made up of 9 angle sections, each 10m in length. The legs and horizontal bracing of the tower were modeled using different size angles specified in bellow

Member	Properties
Vertical member	A200*200*15
	A150*150*16
	A150*150*12
Diagonal bracing	A65*65*6
	A65*65*5
	A50*50*6
Horizontal bracing	A65*65*6
	A50*50*6
	A46*46*6

Table 5.1; Steel sections of Ejere Telecommunication Tower.(Ehiotelecom portal 2014)

5.3.3 Ejerie Tower Loading

The tower is analyzed using loadings from the TIA-222 standards. The tower was loaded first with the TIA-222-C code, which is the code used for the original design of the tower. The tower had discrepancies in member sizes from as built and blueprints, therefore it was necessary to determine if the tower was originally built to standards. It was then loaded with respect to the TIA-222-F code which is the current standard for telecommunication towers. The tower was only analyzed using the TIA-222-F code

LoadPat	Angle Degrees	WindwardCp	LeewardCp	MaxZ m	MinZ m
wind load	0.000	0.800000	0.500000	50.00000	0.00000

LoadPat	WindSpeed meter/sec	TerrainCat	OroFact	TurbFact	StructFact	AirDensity
wind load	22.000	II	1.000000	1.000000	1.000000	1.250000

Table 5.2 Auto wind loads of Ejere Tower generated from Sap2000

5.3.4 Tower Analysis

After the tower was modeled, it was analyzed using the TIA-222-C and TIA-222- F standards, respectively. For the stress checks in each member, the ratios of the actual versus allowable loads and pressures were used. The equation that follows was used for both the TIA-222-C and TIA-222-F checks.

$$\frac{\text{Combined Stress Ratio}}{\text{Allowable Stress Ratio}} < 1.0 \qquad (4.4)$$

From this check, a critical member can be selected for each type of component. The actual combined stress ratios can be divided by the allowable stress ratios (ASR) to determine the percent capacity of each section. This capacity is what determines if the section, and eventually the tower, passes or fails. Since all the section members are adequate to with stand the wind load the combined stress ratio is greater than the allowable stress ratio then all the members pass

Calculation of wind loads on towers were carried out according to ANSI/TIA-222-G-2005[1] for the design wind speed of 22m/s (79.2km/h) , which is the recommended design wind speed for Zone II Normal structures condition according to Ethiopian new code.

5.3.5 Summary of Results

The tower was analyzed for wind loadings using SAP2000. It was found that the towers do not drift laterally in both load combinations one and two. Accordingly it is the diagonal bracing on the tower which controls the stability of the tower in the case of wind loading.

Joint	OutputCase	U1 m	U2 m	U3 m	R1 Radians	R2 Radians	R3 Radians
1	DEAD	0.000000	0.000000	0.000000	-0.000079	0.000079	-1.326E-19
1	EQx	0.000000	0.000000	0.000000	0.000065	-0.000026	-3.962E-06
1	EQy	0.000000	0.000000	0.000000	0.000026	-0.000065	3.962E-06
1	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
2	DEAD	-1.735E-06	-1.735E-06	-0.000349	0.000059	-0.000059	-1.508E-19
2	EQx	0.000081	1.980E-07	0.000340	-0.000122	0.000090	-7.812E-07
2	EQy	1.980E-07	0.000081	0.000340	-0.000090	0.000122	7.812E-07
2	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
3	DEAD	3.535E-07	1.359E-19	-0.000017	1.266E-18	-0.000059	-4.759E-20
3	EQx	0.000075	8.076E-18	-2.026E-06	5.773E-17	0.000090	-7.891E-18
3	EQy	-9.322E-18	0.000075	0.000000	-0.000174	-2.678E-17	-2.613E-07
3	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
4	DEAD	-1.542E-19	3.535E-07	-0.000017	0.000059	-4.389E-19	-3.068E-20
4	EQx	0.000075	-9.048E-18	-2.678E-20	2.742E-17	0.000174	2.613E-07
4	EQy	2.775E-19	0.000075	-2.026E-06	-0.000090	-5.727E-17	4.178E-18

4	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000

Joint	OutputCase	U1 m	U2 m	U3 m	R1 Radians	R2 Radians	R3 Radians
5	DEAD	0.000000	0.000000	0.000000	0.000079	0.000079	5.421E-20
5	EQx	0.000000	0.000000	0.000000	-0.000065	-0.000026	3.962E-06
5	EQy	0.000000	0.000000	0.000000	0.000026	0.000065	3.962E-06
5	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
6	DEAD	-1.735E-06	1.735E-06	-0.000349	-0.000059	-0.000059	4.150E-20
6	EQx	0.000081	-1.980E-07	0.000340	0.000122	0.000090	7.812E-07
6	EQy	-1.980E-07	0.000081	-0.000340	-0.000090	-0.000122	7.812E-07
6	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
7	DEAD	6.237E-20	-3.535E-07	-0.000017	-0.000059	-3.982E-19	-4.460E-20
7	EQx	0.000075	-9.450E-18	0.000000	2.698E-17	0.000174	-2.613E-07
7	EQy	-1.873E-17	0.000075	2.026E-06	-0.000090	-5.620E-17	4.692E-18
7	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
8	DEAD	0.000000	0.000000	0.000000	-0.000079	-0.000079	8.132E-20
8	EQx	0.000000	0.000000	0.000000	-0.000065	-0.000026	-3.962E-06
8	EQy	0.000000	0.000000	0.000000	0.000026	0.000065	-3.962E-06
8	wind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
9	DEAD	1.735E-06	-1.735E-06	-0.000349	0.000059	0.000059	2.711E-20

Table 5.3: Joint Displacements(see Appendex B)

5.4 Analysis under Seismic Loading

The tower is located in southern part of Ethiopia zone for of the seismic map of the country which is in the rift valley of Africa. This area is seismically active therefore a seismic analysis is also needed to determine the stability of the tower. The seismic analysis was done using SAP v9.0. SAP2000 is a static and dynamic structural analysis program that includes linear and non-linear analysis capabilities. Seismic analysis can be performed using SAP 2000 and the ground motion can be modeled using spectrum. Of particular interest for this project were the dynamic modeling capabilities, which can be performed using response spectrum analysis, and combinations of loading scenarios. Modal analysis was performed using Eigenvector analysis for response spectrum. SAP allows the user to input the response spectrum function. Preprocessing in SAP utilizes a graphical interface for defining the tower geometry and properties of members and for defining loads and load combinations. Post processing provides output for internal forces and moments, displacements, mode shapes, and design checks.

5.5 Tower Model Details for seismic loading

The tower was modeled using the information provided by structural tower drawings (Appendix A). Leg, diagonal and horizontal bracings were used as provided in the drawings. Selected element sizes were verified by field measurements.

Tower attachments as shown in the drawings were included in the model. The attachments on the existing tower (such as additional antennas) were not modeled in this analysis, since no information was available at the time. Design drawings were modeled by SAP 200 to predict the response under earthquake loads.

The tower was modeled as a frame structure made up of 9 sections, each 5 m in length. The legs and the diagonals were modeled as angle elements.

5.6 Summary of Results

The tower was analyzed for wind loadings using SAP2000. It was found that the towers do not drift laterally in both load combinations one and two. Accordingly it is the diagonal bracing on the tower which controls the stability of the tower in the case of wind loading.

Joint	OutputCase	U1	U2	U3	R1	R2	R3
		m	m	m	Radians	Radians	Radians
1	DEAD	0.000000	0.000000	0.000000	-0.000079	0.000079	-1.326E-19
1	EQx	0.000000	0.000000	0.000000	0.000065	-0.000026	-3.962E-06
1	EQy	0.000000	0.000000	0.000000	0.000026	-0.000065	3.962E-06
1	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
2	DEAD	-1.735E-06	-1.735E-06	-0.000349	0.000059	-0.000059	-1.508E-19
2	EQx	0.000081	1.980E-07	0.000340	-0.000122	0.000090	-7.812E-07
2	EQy	1.980E-07	0.000081	0.000340	-0.000090	0.000122	7.812E-07
2	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
3	DEAD	3.535E-07	1.359E-19	-0.000017	1.266E-18	-0.000059	-4.759E-20
3	EQx	0.000075	8.076E-18	-2.026E-06	5.773E-17	0.000090	-7.891E-18
3	EQy	-9.322E-18	0.000075	0.000000	-0.000174	-2.678E-17	-2.613E-07
3	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
4	DEAD	-1.542E-19	3.535E-07	-0.000017	0.000059	-4.389E-19	-3.068E-20
4	EQx	0.000075	-9.048E-18	-2.678E-20	2.742E-17	0.000174	2.613E-07
4	EQy	2.775E-19	0.000075	-2.026E-06	-0.000090	-5.727E-17	4.178E-18
4	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
5	DEAD	0.000000	0.000000	0.000000	0.000079	0.000079	5.421E-20
5	EQx	0.000000	0.000000	0.000000	-0.000065	-0.000026	3.962E-06
5	EQy	0.000000	0.000000	0.000000	0.000026	0.000065	3.962E-06
5	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
6	DEAD	-1.735E-06	1.735E-06	-0.000349	-0.000059	-0.000059	4.150E-20
6	EQx	0.000081	-1.980E-07	0.000340	0.000122	0.000090	7.812E-07
6	EQy	-1.980E-07	0.000081	-0.000340	-0.000090	-0.000122	7.812E-07
6	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
7	DEAD	6.237E-20	-3.535E-07	-0.000017	-0.000059	-3.982E-19	-4.460E-20
7	EQx	0.000075	-9.450E-18	0.000000	2.698E-17	0.000174	-2.613E-07
7	EQy	-1.873E-17	0.000075	2.026E-06	-0.000090	-5.620E-17	4.692E-18
7	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
8	DEAD	0.000000	0.000000	0.000000	-0.000079	-0.000079	8.132E-20
8	EQx	0.000000	0.000000	0.000000	-0.000065	-0.000026	-3.962E-06
8	EQy	0.000000	0.000000	0.000000	0.000026	0.000065	-3.962E-06
8	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
9	DEAD	1.735E-06	-1.735E-06	-0.000349	0.000059	0.000059	2.711E-20

Table 5.4: Joint Displacements(see Appendex B)

5.7 Analysis under Seismic Loading

The tower is located in southern part of Ethiopia zone of the seismic map of the country which is in the rift valley of Africa. This area is seismically active therefore a seismic analysis is also needed to determine the stability of the tower. The seismic analysis was done using SAP v9.0. SAP2000 is a static and dynamic structural analysis program that includes linear and non-linear analysis capabilities. Seismic analysis can be performed using SAP 2000 and the ground motion can be modeled using response spectrum. Of particular interest for this project were the dynamic modeling capabilities, which can be performed using response spectrum analysis, and combinations of loading scenarios. Modal analysis was performed using Eigenvector analysis method. SAP2000 allows the user to input the response spectrum function. Preprocessing in SAP2000 utilizes a graphical interface for defining the tower geometry and properties of members and for defining loads and load combinations. Post processing provides output for internal forces and moments, displacements, mode shapes, and design checks.

5.8 Tower Model Details for seismic loading

The tower was modeled using the information provided by structural tower drawings (Appendix A). Leg, diagonal and horizontal bracings were used as provided in the drawings. Selected element sizes were verified by field measurements.

Tower attachments as shown in the drawings were included in the model. The attachments on the existing tower (such as additional antennas) were not modeled in this analysis, since no information was available at the time. Design drawings were modeled by SAP 200 to predict the response under earthquake loads.

The tower was modeled as a frame structure made up of 9 sections, each 5m in length. The legs and the diagonals were modeled as angle elements.

5.9 Response spectrum case load assignments

G.W.Housner was instrumental in wide spread acceptance of the concept of the earth quack response spectrum-Introduced by M.A.Biot in 1932 as a practical means of characterizing ground motion and their effects on the structures.Now acenteral in earth quack engineering,the response spectrum provides a convenient means to summarize the pick response of all possible linear SDF systems to aparticular component of the ground motion. It also provides a practical approach to apply the knowledge of structural dynamics to the design of structures and development of lateral force required in building codes. Sap2000 automaticaly generate the spectrum parameters that can help the user to discuss the results. Accordingly all the possible values of category 2(damping ratio etc) are provided to the software so as to make analysis.

Name	Period Sec	Accel	FuncDamp
UNIFRS	0.000000	1.000000	0.050000
UNIFRS	1.000000	1.000000	

Table 5.5: Function - Response Spectrum

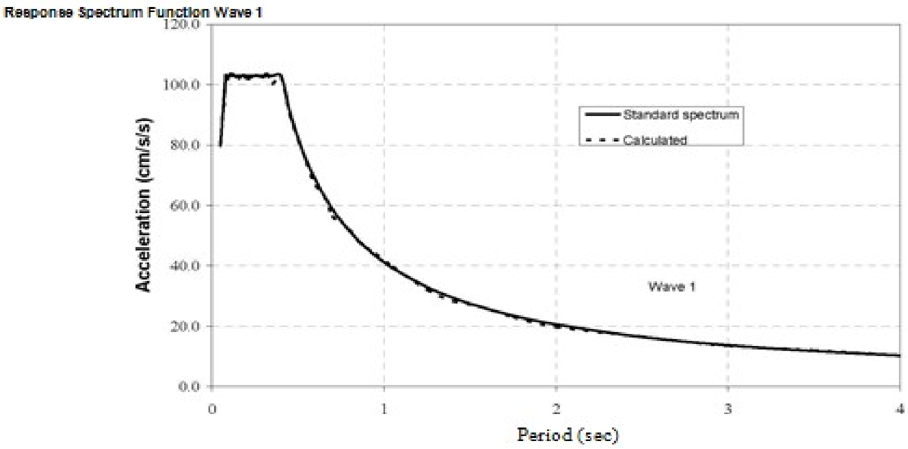


Figure 5.3 Response spectrums for wave 1

5.10 Base shear of the Tower

Base shear scaling is a procedure required by S1170.5 as a means to ensure that the minimum strength of a structure designed using the Modal Response Spectrum method (MRS) is similar to the strength that would be required if the structure was designed using the Equivalent Static Method (ESM). The inclusion of this provision in the codecan be seen from a historic perspective. In the 1976 and 1984 versions of the Loadings Standard (NZS4203), a Modal Response Spectrum analysis was a solution method that the designer could elect to use and obtain, what is considered a more accurate distribution of strength throughout the structure.

The reward for using the MRS method being a reduced base shear equal to 90% of the value calculated using the Equivalent Static Method. Accordingly the following results from Sap shows how the Tower reacts to the base shear.

Table 5.6: Base Reactions

Table 17: Base Reactions

OutputCase	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m
DEAD	-2.331E-15	1.556E-14	129.774	-1.007E-12	-3.286E-13	3.020E-14
EQx	-9.733	2.215E-12	1.723E-13	-9.643E-11	-315.5021	4.963E-12
EQy	2.210E-12	-9.733	-4.860E-13	315.5021	9.624E-11	-2.713E-12
w ind load	0.000	0.000	0.000	0.0000	0.0000	0.0000

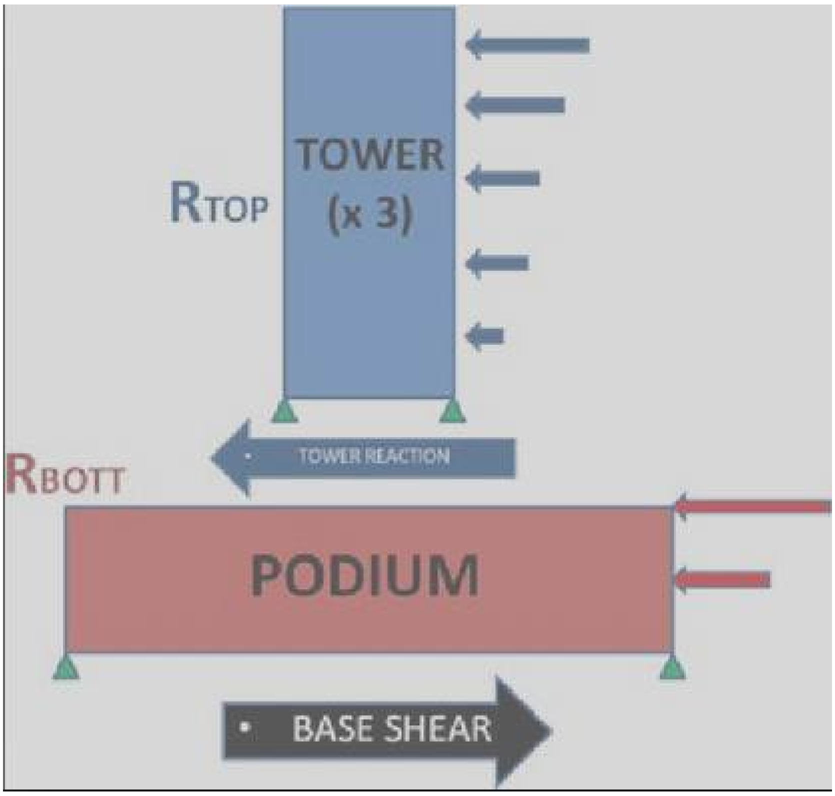


Figure 5.4 Schematic diagram which shows base shear of the Tower

5.11 Design spectrum

A response spectrum is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency, that are forced into motion by the same base vibration or shock.

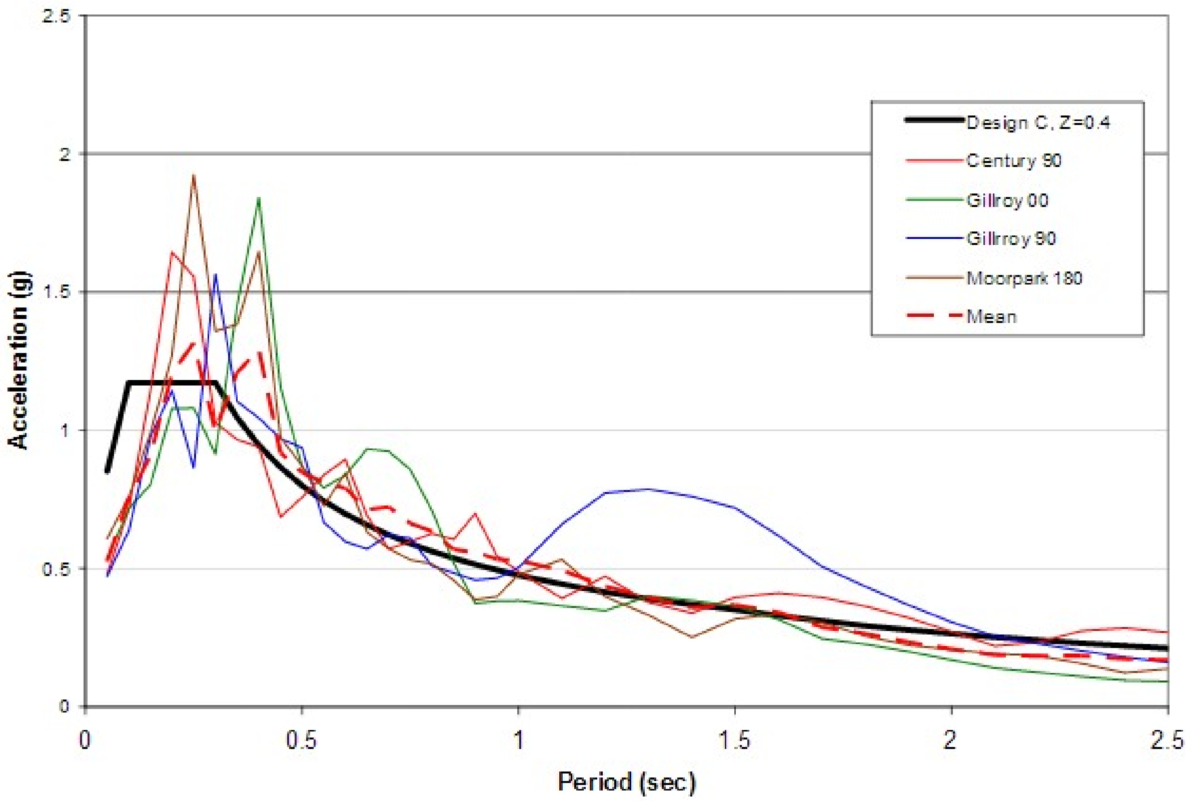


figure 4.5 Design response spectrum of a Tower(taken from google)

5.12 Results from Seismic Analysis

5.12.1 Mode shapes

A total of sixteen mode shapes were selected for the analysis. Selected modes are shown in Appendix. The modal analysis was used for the dynamic analysis using the response spectrum loading function as described earlier.

Detailed evaluation of the results of the SAP 2000 analysis revealed that the braces were stressed the most when compared to the leg and cable members. The critical braces were defined using the calculated brace axial forces from modal analysis for the standard spectrum function, modal analysis for the time history function, spectrum function combination, and time history function combination for maximum and minimum values.

Axial forces for all modes are summarized in Appendix B. The maximum results from all combinations are shown in Appendex, which indicates the location of the most critical braces in the tower are at or near the tower legs.

Supports reactions, maximum axial forces in leg members and maximum horizontal deflections of each tower with respect to the load combination describe above were obtained from SAP 2000 analysis results of respective tower models. Figure 2, 3 and 4 show the maximum uplift, downward and horizontal reactions in towers respectively. As expected, maximum uplift reactions in each and every case are observed when dead load has a factor of safety of 0.9, while maximum downward and horizontal reactions are observed when dead load has a factor of safety of 1.2. According to results of the graphs, support reactions under assumed earthquake loading condition for Etiopia are very much higher than the support reaction under design wind loading .Values under wind loading and earthquake loading increases with the increase of the tower height. Accordingly, there is uplift reaction under assumed earthquake loading condition, tower almost reach to the design support reaction condition if it has been designed considering design wind speed 22 m/s. These critical braces were evaluated further in the parametric study described later in this section of the report

5.12.2 Condition Indexing System

A condition indexing (CI) system is a methodology used to systematically quantify a structure's physical condition. The CI ranges from 0 to 100 with 0 being the worst possible condition and 100 being the best. CI systems are very valuable tools for complex, networked facilities such as the MODOT communication tower system. To help maintain the structural integrity of the tower system, a function-based condition indexing system for communication towers was developed (Tulasi, 2005) and is summarized in this section. This system has a series of steps used to eventually determine how a tower will react in an emergency situation and also prioritize the towers in terms of need of repair.

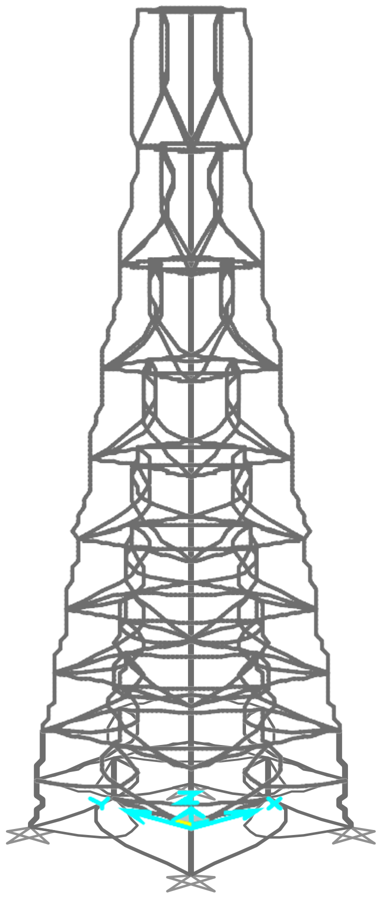


Figure 6.1: Deformed shape of the Tower

5.12.3 The Wind loading indexing

The tower was first evaluated using SAP2000 software under wind loading for various assumed damage (deterioration) levels. Damage levels were assumed to range from 0%, or no damage, to 50% damage.

5.12.4 Effects of Bracing Damage

Simulated deterioration was introduced to the bracing by decreasing the cross sectional area from0% to 50% by 10% increments. The tower was loaded as mentioned previously for all cases of deterioration. Tables 6.4a, 6.4b and Figure 6.4 show how the legs and bracing reacted to the damage of the braces. It can be seen in Figure 6.4(d) that the braces reach 100% capacity before the braces are damaged 10%. This shows that the braces are critical to the structural integrity of the tower.

The legs were affected similarly due to the percent damage increase of the bracing. As the brace damage was increased the legs percent capacities were relieved by 3.5%, respectively. These are very small decreases in capacities, which are believed to be related to the loading of the tower. As the cross sections of the braces are decreased, the area to which the wind loadings are applied decreases, therefore making the total load on the tower slightly decrease. In reality, there would not be much decrease in the area to which the load is applied since rust coats the member. There would be a decrease in area giving strength to the tower, as rust is very weak, but the area to which loads are applied would not decrease. For this research the cross sections are decreased as it is a simple way to account for deterioration

CHAPTER SIX

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

As per the objective of this research, performance of the existing towers (which is originally not designed for earthquake loading) were analyzed considering different earthquake loading as per equivalent static method given in ANSI/TIA-222-G[1] for selected four legged tower. According to findings of this study, it quite evident that four legged tower in the height of 50m will survive without any problem under wind speed of 22m/s (which is the most probable magnitude for wind load that can occur in Ethiopia), if such towers have been properly designed for recommended earth quake of the respective zones. Even under sever or very severe wind speed loading conditions, all of the tower member will behave satisfactorily, if such towers have been designed considering a designed wind speed of 22m/s. However, under a very severe wind loading condition, towers may have almost reached to the designed stress state if such towers have been designed considering design wind speed greater than what the researcher considered.

In the other words if you consider the software outputs in the appendix the tower joints deflects laterally and rotates in all output cases(Dead, EQX and EQY) more than when there is static wind load is applied to the model. Accordingly all 84 joints displaces and rotates 0 m and 0 radian respectively in all wind load cases but due to dead load outp ut cases the assigned joints displaces(U1) maximum of $1.735*10^{-6}$m in positive direction on joint number 9 and $-1.735*10^{-6}$m in negative direction on joint number 6 . Maximum Displacement (U2) due to dead load is on joint 6 which is $1.735*10^{-6}$ m in positive direction and joint 9 which is $-1.735*10^{-6}$ m in negative direction. Maximum displacements (U3) is -0.000912m in negative direction in joint 21,24 and the joints do not displace in positive direction in these scenario. From these all the researcher can conclude that due to dead loads the joints in the lower parts of the Tower displaces more than the joints in the tops of the Tower. When we come to earth quack in X direction the tower displaces maximum of 0.031656m on joints(77 to 84) for U1 and $8.970*10^{-8}$m in positive direction U2 for joint number (74) and $-6.566*10^{-8}$m in joint(76) in the negative direction and for U3 due to EQX the tower maximally displaces 0.0005m in joints (77,78 and80) from these the writer concludes that the earth quack in the X direction affects the Top parts of the Tower than any other parts more. Finally when we come to the effect of earth quack in the y direction maximum U1 is $2.264*10^{-9}$m and -8.970*

10^{-8}m and U2 is 0.031656 on joint 82, up on these the writer concludes that the upper part of the tower is

more affected by the lateral earth quack load in the Y direction than any other parts of the Tower.Finally the writer concludes that joint displacements taken place because of the lateral earth quack load and the inertia of the structure. Since inertia is out of the scope of these study the earth quaks governs the joint displacement designs of the Tower than the wind loads in particular case of the thesis.

The joints of the Tower reacts on the earth quack loads than the wind loads according to these study, up on which due to dead load joint 1 is f1=1.872kn,f2=1.872kn and f3=32.443kn, due to EQX joint 1 is f1= -2.433kn,f2=-1.423kn and f3=-30.932kn and reaction of joint 1 due to EQY is f1=-1.423kn,f2=-2.433kn and f3=-30.932kn and the tower reacts 0kn in all to the wind loads. From these regardless of dead loads the earth quack governs the

Not only that but also there is a base shear reactions on the software output indicating that almost no base shear reaction scenarios in both load combinations of the wind and dead load but we can easily take a look at the values when loaded with the remaining earth quakes with dead load scenarios.

Finally based on the above findings the researcher concludes that it should be customary to design earth quack loads than wind loads for the towers located in seismic zones of Ethiopia despite Ethiopia is not prone to seismic phenomena's lands like Japan and USA, which means it is the earth quaks that governs the design in these specific study.

Recommendations

To create more accurate results for the towers design and analysis, a few topics need to be addressed further. Following are several recommendations for the future that could make the results more complete.

- SAP2000 was limited in that it was unable to create cable members. If there is any interested person to conduct his/her study on the Telecom Towers the guys should be modeled as beams and had compressive axial forces present. The compressive axial forces were neglected from the results. A better way of modeling the tower could be found, possibly using a different modeling tool that would give more accurate results when the tower is subjected to seismic loadings.
- In these study rotation of the joints are not interested by the researcher so any person can take these parameter and conduct an experiment on these and these paper can give a good clue.

Finally the researcher invites any researcher to take an experiment on Telecom Towers located in different parts of the country with all different parameters the researcher used in this particular thesis and add something special.

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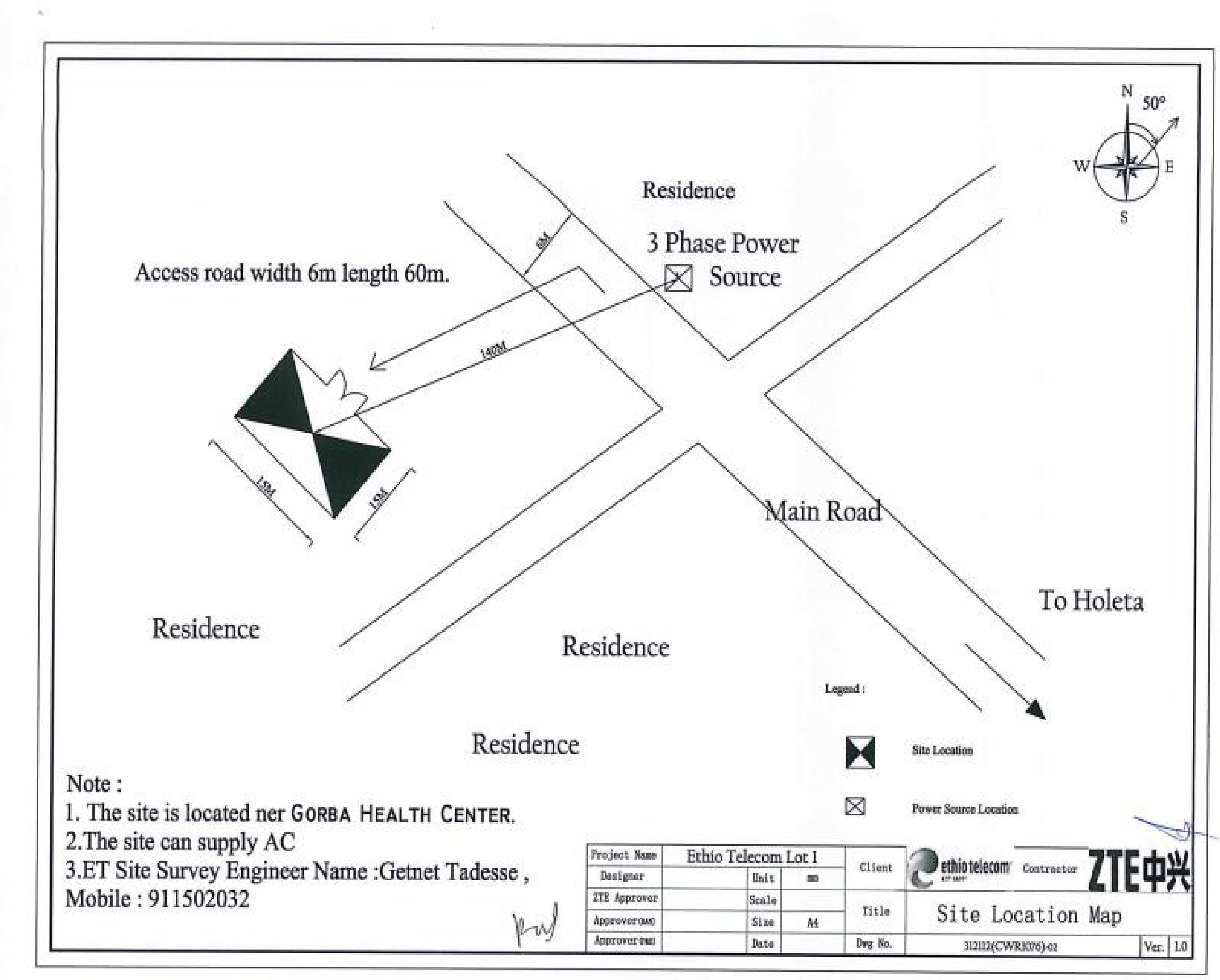
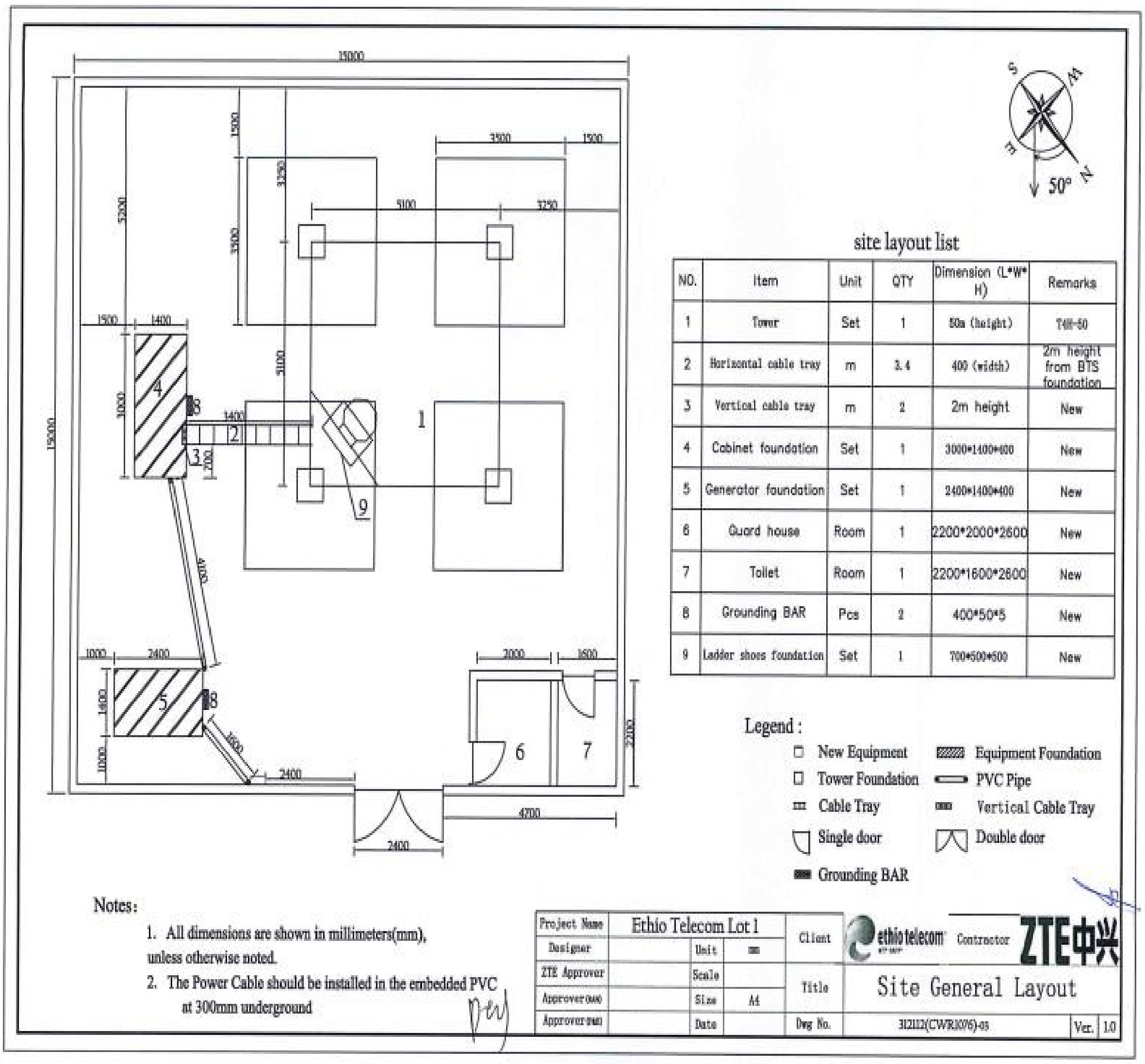
APPENDIX A: DETAILED DESIGN DRAWING

Detailed Design Drawings

Site ID:	312112(CWR1076)	Site Name:	Ejerie_02
Latitude:	9.15479°	Longitude:	38.32133°
Site type:	Green field	Tower:	T4H-50
Address:	Oromiya,West Shoa Zone,Ejerie Wereda, Gorba Kebele,Gorba Health Center.(CWR)		

Project Name	Ethio Telecom Lot 1			Client	ethio telecom
Designer		Unit	mm	Contractor	ZTE中兴
ZTE Approver		Scale		Title	Site General Layout
Approver		Size	A4		
Approver		Date		Dwg No.	312112(CWR1076)-01 Ver. 1.0


N
50°
W
E
S
Residence
3 Phase Power
Source
Access road width 6m length 60m.
6M
140M
15M
15M
Main Road
To Holeta
Residence
Residence
Residence
Legend :
Site Location
Power Source Location
Note :
1. The site is located ner GORBA HEALTH CENTER.
2.The site can supply AC
3.ET Site Survey Engineer Name :Getnet Tadesse ,
Mobile : 911502032
Project Name
Ethio Telecom Lot 1
Designer
Unit
mm
ZTE Approver
Scale
Size
A4
Date
Client
ethio telecom
Contractor
ZTE中兴
Title
Site Location Map
Dwg No.
Ver.
1.0



15000
3500
1500
5100
3250
site layout list
NO. | Item | Unit | QTY | Dimension (L*W*H) | Remarks
1 | Tower | Set | 1 | 50m (height) | T4H-50
2 | Horizontal cable tray | m | 3.4 | 400 (width) | 2m height from BTS foundation
3 | Vertical cable tray | m | 2 | 2m height | New
4 | Cabinet foundation | Set | 1 | 3000*1400*400 | New
5 | Generator foundation | Set | 1 | 2400*1400*400 | New
6 | Guard house | Room | 1 | 2200*2000*2600 | New
7 | Toilet | Room | 1 | 2200*1600*2600 | New
8 | Grounding BAR | Pcs | 2 | 400*50*5 | New
9 | Ladder shoes foundation | Set | 1 | 700*500*500 | New
Legend :
New Equipment
Equipment Foundation
Tower Foundation
PVC Pipe
Cable Tray
Vertical Cable Tray
Single door
Double door
Grounding BAR
Notes:
1. All dimensions are shown in millimeters(mm), unless otherwise noted.
2. The Power Cable should be installed in the embedded PVC at 300mm underground
Project Name
Ethio Telecom Lot 1
Designer
Unit
mm
ZTE Approver
Scale
Approver
Size
A4
Approver
Date
Client
ethio telecom
Contractor
ZTE中兴
Title
Site General Layout
Dwg No.
312112(CWR3076)-05
Ver.
1.0

CIVIL WORK GENERAL NOTES

GENERAL

1.The design of tower foundation is according to ANSI/TIA/EIA-222F-1996.
All reinforced concrete design is according to ACI318M-08;Structural steel design is according AISC recommendations;Structural design is according to ANSI-TIA-222-G-2005.

2.All dimensions are in millimetres and all level are in metres above arbitrary datum
Structure steel design is according to AISC recommendations
unless otherwise noted

3.The contractor is responsible for the dimensional accuracy and the correct setting out of all work on site

4.The contractor is responsible for checking all site sizes prior to commencement of work

5.Prior to commencement of work on site the contractor must give reasonable notice to the owener and/or tenant of the property

GREEN FILED SITES

BACKFILLING

1.Back fill shall be done in layers not exceeding 150mm measured as compacted
Material and saturated with AASH.TO sufficient water and compacted to produce in-situ
density not less than 95% of the max modified dry density ,or as shown in the foundation drawing

2.Excavation&filling back for pit of foundation shall follow the requirments of geotechnical investigation report

3.finished ground level must be 300mm higher than existing ground level or road whichever is higher with 200 compacted soil layer plus 100mm compacted gravel cover

BACKFILL MATERIAL

1.Any materials:the condition and character prevent which their use for construction purposes in conformity with the specifications ,such as soaked or softened soil,frozen soil,snow ,ice, waste tree stumps,mud and peat ,organic soil, silt and swelling clays ET.shall not be used;

2.Any stones ,blocks and lumps of clay of a dimension exceeding half of the layer-thickness built in (maximum 200mm thick layers)shall not be used for construction purposes

4.The filling material shall be spread in uniform layers,the thickness of such layers to be fixed during the execution in relation to the compaction equipment used ,in such a manner that a compaction higher is attained throughout the entire thickness of the layer

COMPACTION

1.Each individual layer shall be compacted throughout the full theoretical width of the filling

and subsequent removal of soil

2.After compaction of the materials the contractor must provide documentation of the reached density of the compated materials.the documentation of the density of the compacted materials shall consist of at least 2 sand replacement tests per footing carried out in accordance to
ASTM,designation:D4919-89,standard test methods for density of soil and rock in place by the sand replacement

EQUIPMENT FOUNDATION CONSTRUCTION

1.All the equipment foundation ,such as shelter foundation and outside cabinet foundation,shalll be casted on solid foundation soil.Before cating concrete,mantle rock on the earth surface shall be replaced by 300mm gravel or rammed clay;

2.outside part of foundation shall be plastered 15mm 1:2.5 cement mortar;

3.The curing period of concrete foundation shall be no less than 7 days;

TOWER FOUNDATION CONSTRUCTION

1.The curing period of tower foundation concrete shall be no less than 7 days ; After the strength of concrete reach specification in tower foundation darwings,Installing of tower body can begain;

2.Backfill of tower foundation according to specifications mentioned above;

3.outside part of foundation shall be plastered 15mm 1:2.5 cement mortar;

4.The number and fixed position of embedded bolts must be according to tower drawings strictly;The outside part of bolts shall be anti-corrosive treatment or covered by fine stuff concrete.

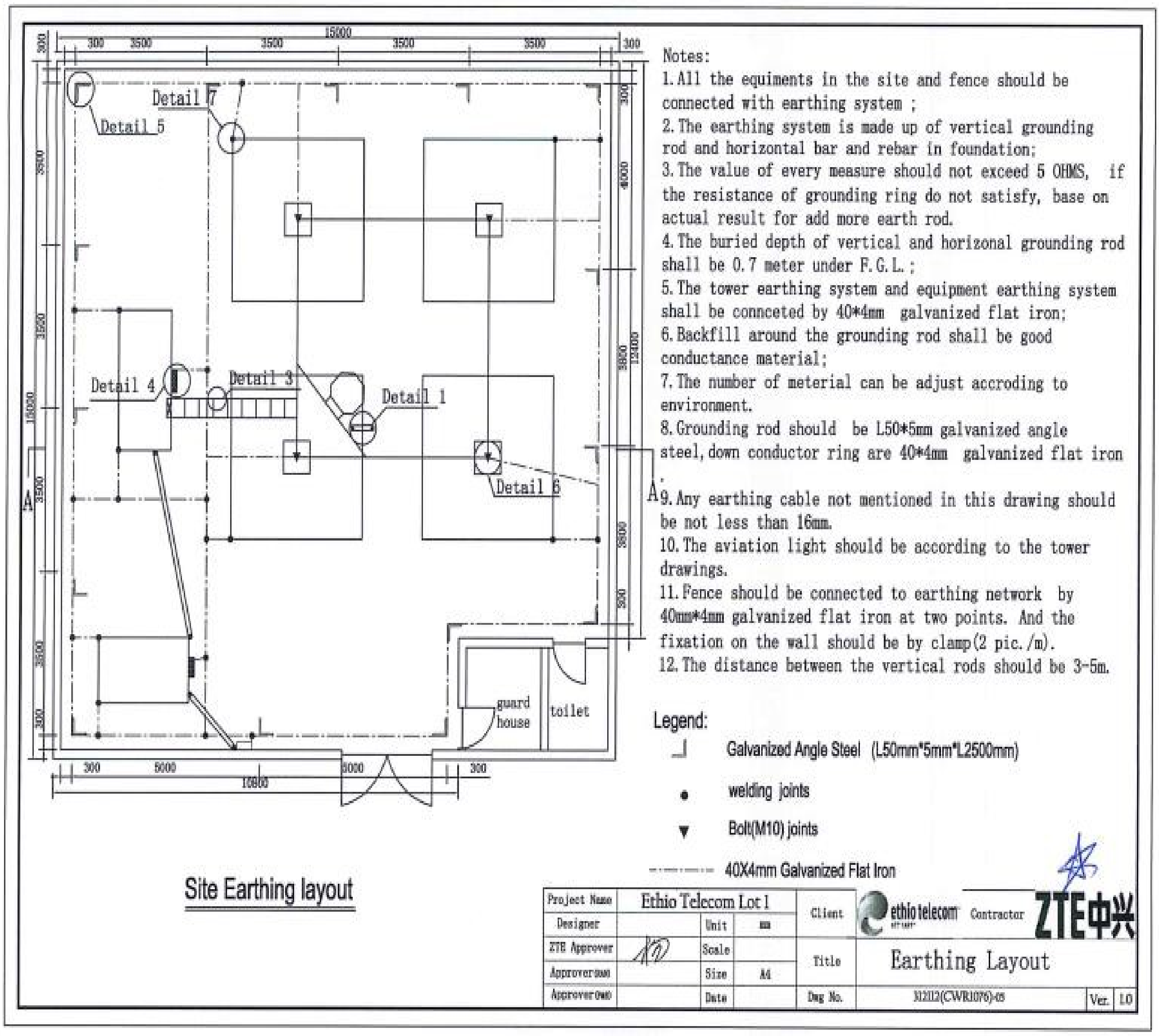
EARTHING WORK

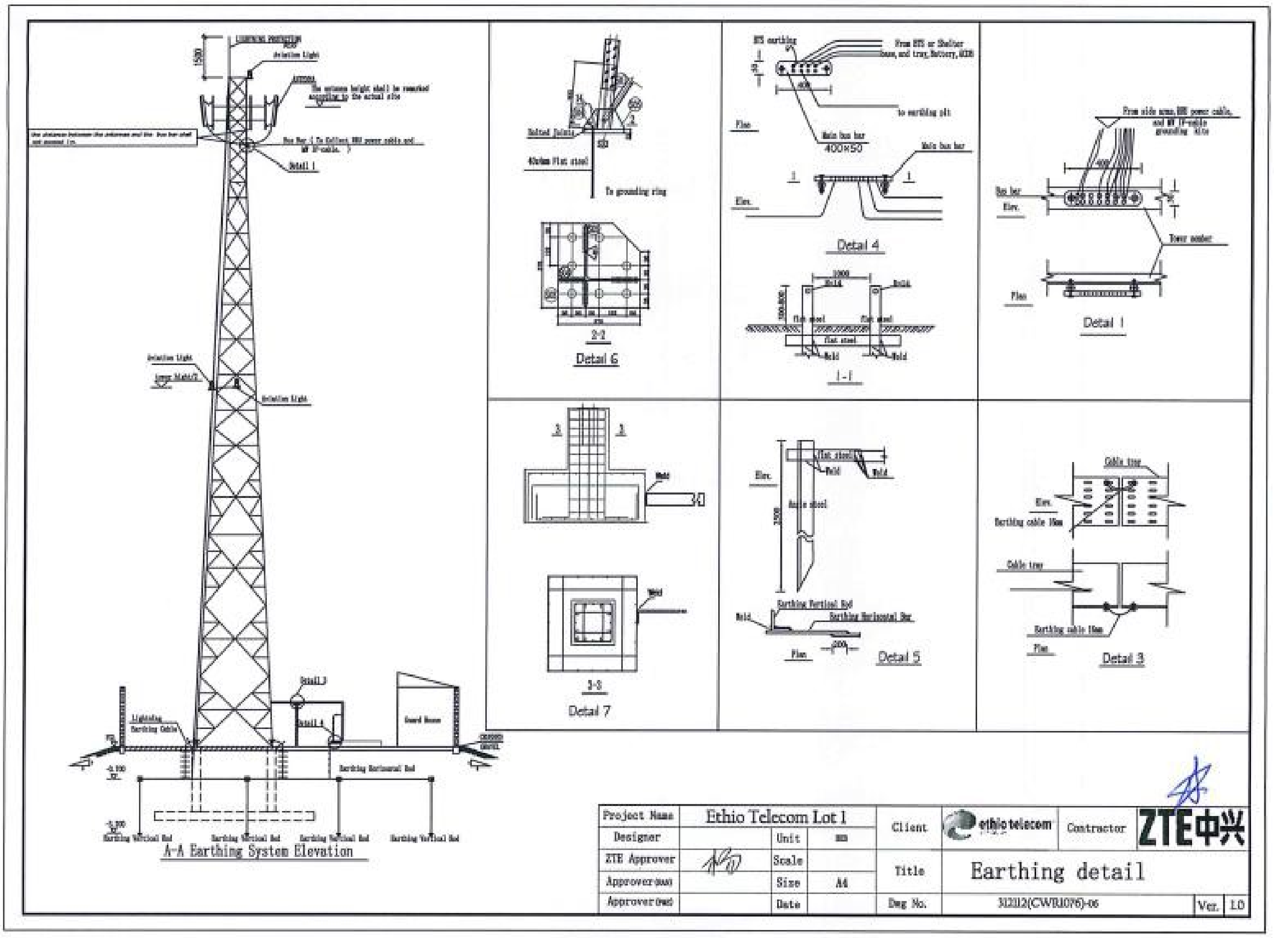
1.Earthing system including vertical and horizonal grounding rod;Vertical grounding rod is 2.5 meters 50X5 angle steel;material of horizonal grounding rod is 40X4 flat steel;which shall be buried no less than 700mm under ground;

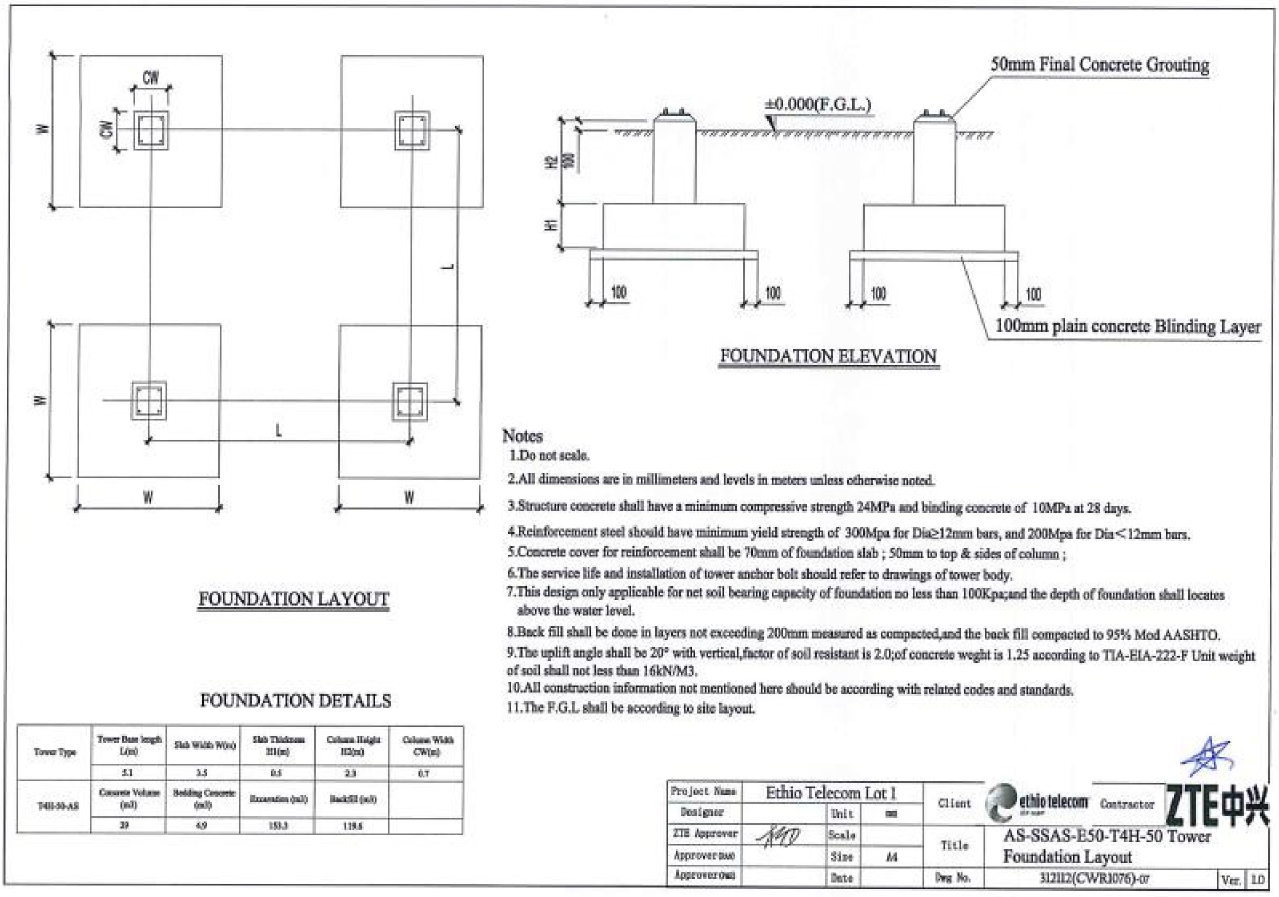
2.While Resistance of earthing system can not meet to code demanding,chemical methods or expanding grounding system shall be adopted;

3.Backfill around the grounding rod shall be good conductance material;gravel and rock is not permited;

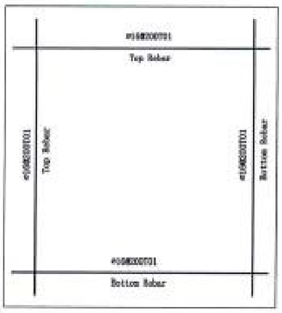
Project Name	Ethio Telecom Lot 1			Client		Contractor
Designer		Unit	mm			
ZTE Approver		Scale		Title	Civil Work General Notes	
Approver		Size	A4			
Approver		Date		Dwg No.	312112(CWR1076)-04	Ver. 1.0


Notes:
1. All the equiments in the site and fence should be connected with earthing system ;
2. The earthing system is made up of vertical grounding rod and horizontal bar and rebar in foundation;
3. The value of every measure should not exceed 5 OHMS, if the resistance of grounding ring do not satisfy, base on actual result for add more earth rod.
4. The buried depth of vertical and horizonal grounding rod shall be 0.7 meter under F.G.L.;
5. The tower earthing system and equipment earthing system shall be connceted by 40*4mm galvanized flat iron;
6. Backfill around the grounding rod shall be good conductance material;
7. The number of meterial can be adjust accroding to environment.
8. Grounding rod should be L50*5mm galvanized angle steel, down conductor ring are 40*4mm galvanized flat iron.
9. Any earthing cable not mentioned in this drawing should be not less than 16mm.
10. The aviation light should be according to the tower drawings.
11. Fence should be connected to earthing network by 40mm*4mm galvanized flat iron at two points. And the fixation on the wall should be by clamp(2 pic./m).
12. The distance between the vertical rods should be 3-5m.
Legend:
Galvanized Angle Steel (L50mm*5mm*L2500mm)
welding joints
Bolt(M10) joints
40X4mm Galvanized Flat Iron
Detail 1
Detail 3
Detail 4
Detail 5
Detail 6
Detail 7
guard house
toilet
Site Earthing layout
Project Name
Ethio Telecom Lot 1
Designer
ZTE Approver
Unit
mm
Scale
Size
A4
Date
Client
ethio telecom
Contractor
ZTE中兴
Title
Earthing Layout
Dwg No.
Ver.
1.0

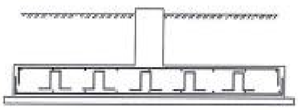

A-A Earthing System Elevation
Detail 6
Detail 4
Detail 1
Detail 7
Detail 5
Detail 3
Project Name
Ethio Telecom Lot 1
Designer
ZTE Approver
Unit
Scale
Size
A4
Date
Client
ethio telecom
Contractor
ZTE中兴
Title
Earthing detail
Dwg No.
312112(CWRJ076)-06
Ver. 1.0



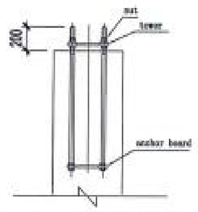
Tower Type	Tower Base length L(m)	Slab Width W(m)	Slab Thickness H1(m)	Column Height H2(m)	Column Width CW(m)
T4H-50-AS	5.1	3.5	0.5	3.3	0.7
	Concrete Volume (m3)	Bedding Concrete (m3)	Excavation (m3)	Backfill (m3)	
	39	4.9	153.3	119.6	



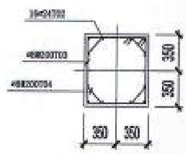
SLAB PLAN



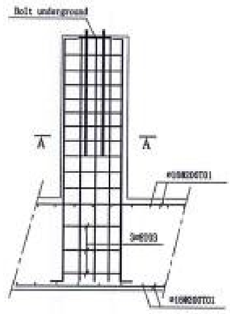
SLAB ELEVATION



ANCHOR BOLT OF FIXING



A-A SECTION



COLUMN REBAR DETAIL

REBAR DETAILS

Tower Type	Bar Mark	Bar Size d(mm)	Bar Spacing (mm)	A (mm)	B (mm)	C (mm)	Unit Length (mm)	Qty.	Total Length (m)	Mass Per Meter (Kg/m)	Total Rebar Mass (Kg)
E50	T01	16	200	320	3400		3960	304	1203	1.58	2589
	T02	24	150	150	2700	300	3030	64	195	2.47	
	T03	8	200	600	600	75	2550	56	143	0.395	
	T04	8	200	201	316	75	2168	56	121	0.395	
	T05	8	600	350	316	420	1822	144	263	0.395	

REBAR SHAPE

BAR MARK	SHAPE	LENGTH
T01	A B A	L=2A+B-5d
T02	A B C	L=A+B+C-5d
T03	A C B	L=2(A+B+C)-7.5d
T04	A B C	L=4(A+B)+2C-7.5d
T05	A B C B C	L=A+2B+2C-10d

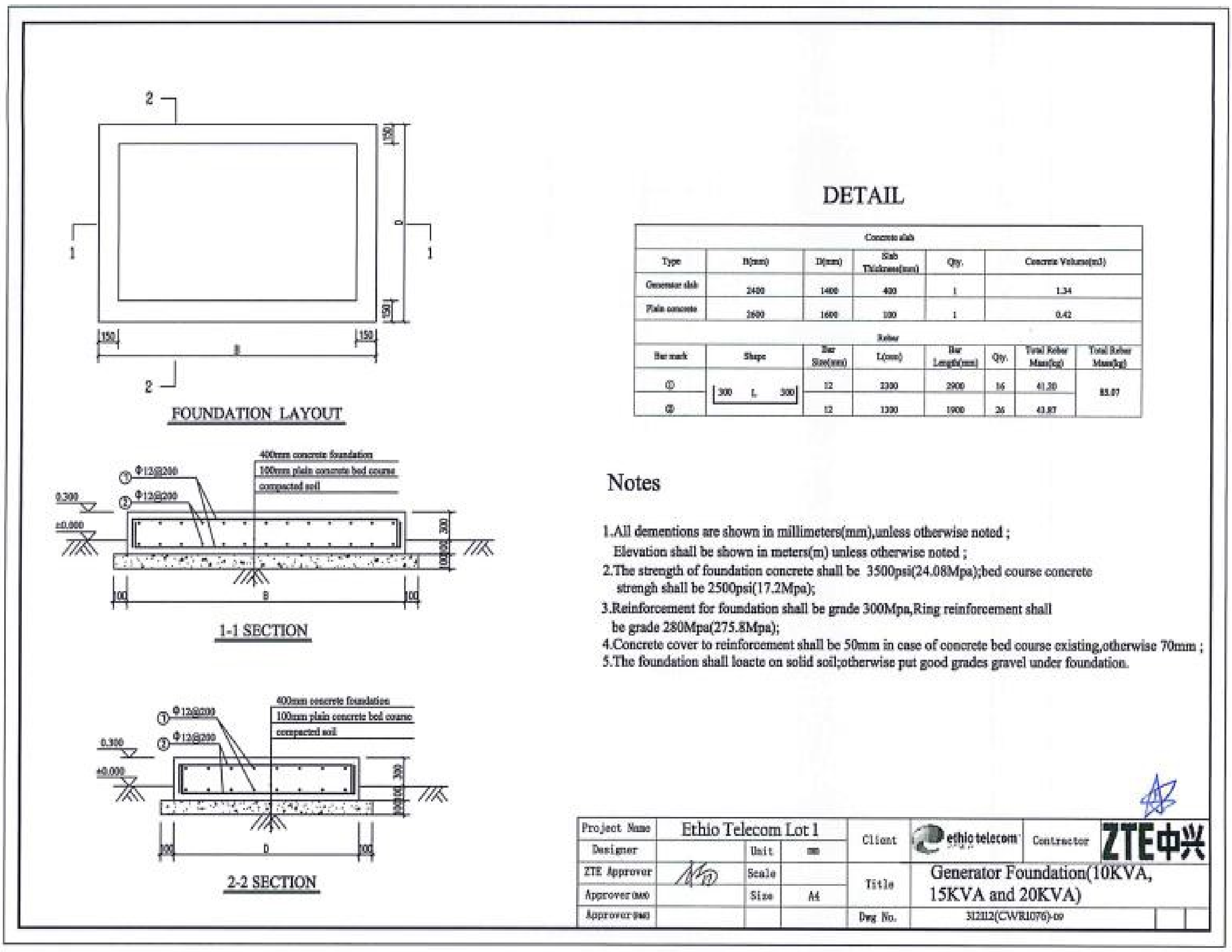
Notes

1.The curring period of foundation shall be no less than 7 days before installing of tower;

2.The position and number of embedded bolts shall be determined according to tower drawings;

3.The outside part of bolts shall be anti-corrosive treatment or covered by fine stuff concrete;

Project Name	Ethio Telecom Lot 1			Client	ethio telecom
Designer		Unit	mm		Contractor ZTE中兴
ZTE Approver		Scale		Title	AS-SSAS-E50-T4H-50 Tower Section Plan
Approver		Size	A4		
Approver		Date		Dwg No.	312112(CWRI076)-08 Ver. 1.0

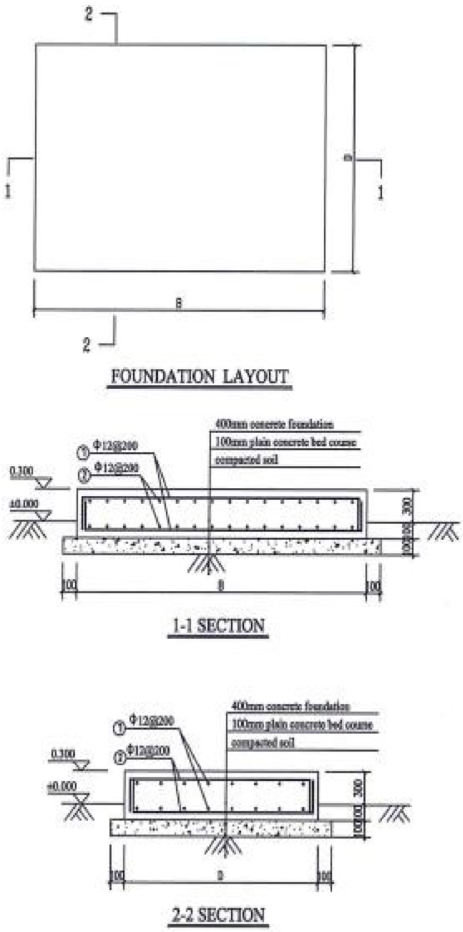


FOUNDATION LAYOUT
1-1 SECTION
2-2 SECTION
400mm concrete foundation
100mm plain concrete bed course
compacted soil
Φ12@200
0.300
±0.000
DETAIL

Concrete slab					
Type	B(mm)	D(mm)	Slab Thickness(mm)	Qty.	Concrete Volume(m3)
Generator slab	2400	1400	400	1	1.34
Plain concrete	2600	1600	100	1	0.42

Rebar							
Bar mark	Shape	Bar Size(mm)	L(mm)	Bar Length(mm)	Qty.	Total Rebar Mass(kg)	Total Rebar Mass(kg)
①	300 L 300	12	2300	2900	16	41.20	85.07
②		12	1300	1900	26	43.87	

Notes
1.All dementions are shown in millimeters(mm),unless otherwise noted ;
Elevation shall be shown in meters(m) unless otherwise noted ;
2.The strength of foundation concrete shall be 3500psi(24.08Mpa);bed course concrete strengh shall be 2500psi(17.2Mpa);
3.Reinforcement for foundation shall be grade 300Mpa,Ring reinforcement shall be grade 280Mpa(275.8Mpa);
4.Concrete cover to reinforcement shall be 50mm in case of concrete bed course existing,otherwise 70mm ;
5.The foundation shall loacte on solid soil;otherwise put good grades gravel under foundation.
Project Name
Ethio Telecom Lot 1
Designer
Unit
mm
ZTE Approver
Scale
Approver
Size
A4
Client
ethio telecom
Contractor
ZTE中兴
Title
Generator Foundation(10KVA, 15KVA and 20KVA)
Dwg No.
312112(CWR1076)-09



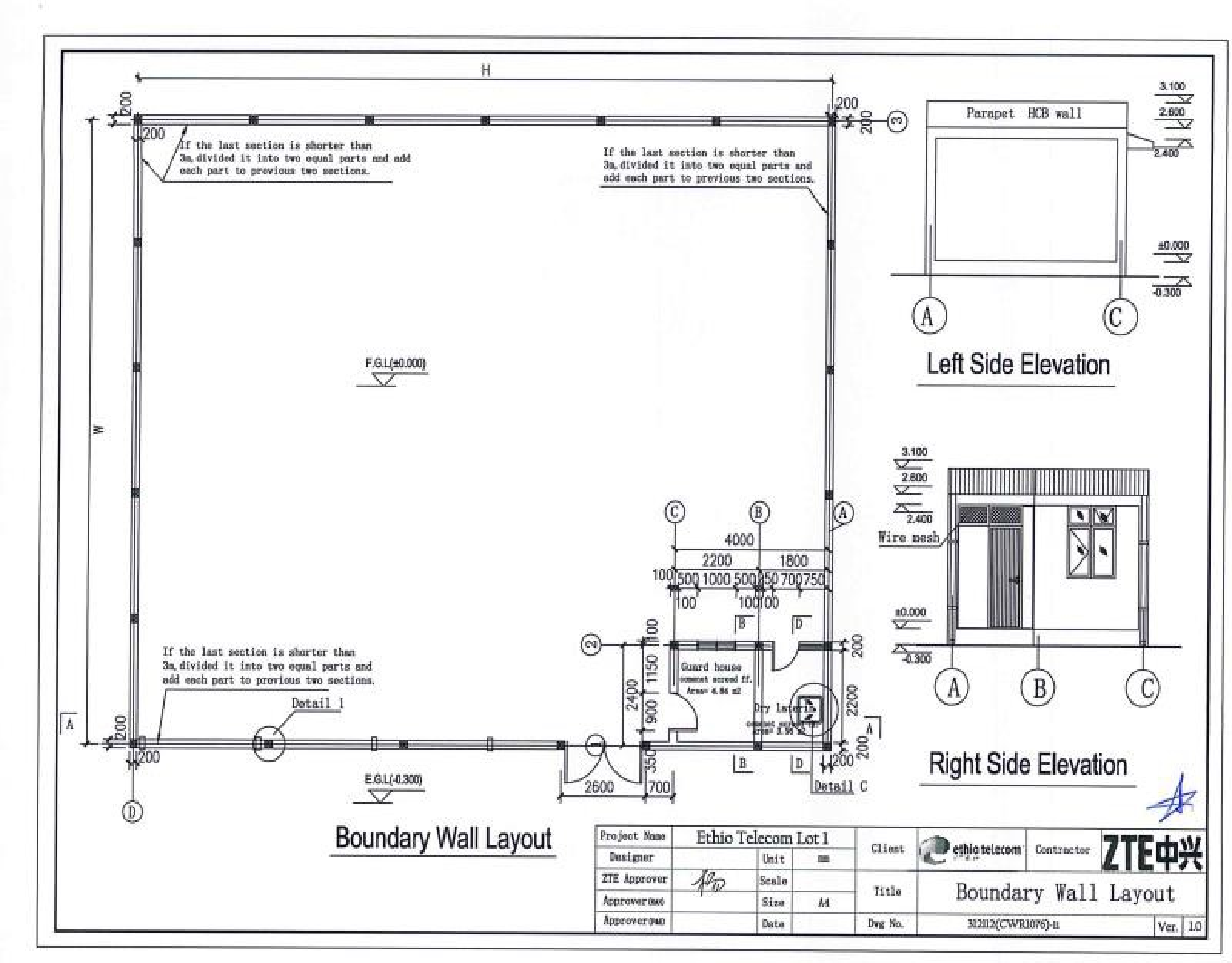
DETAIL

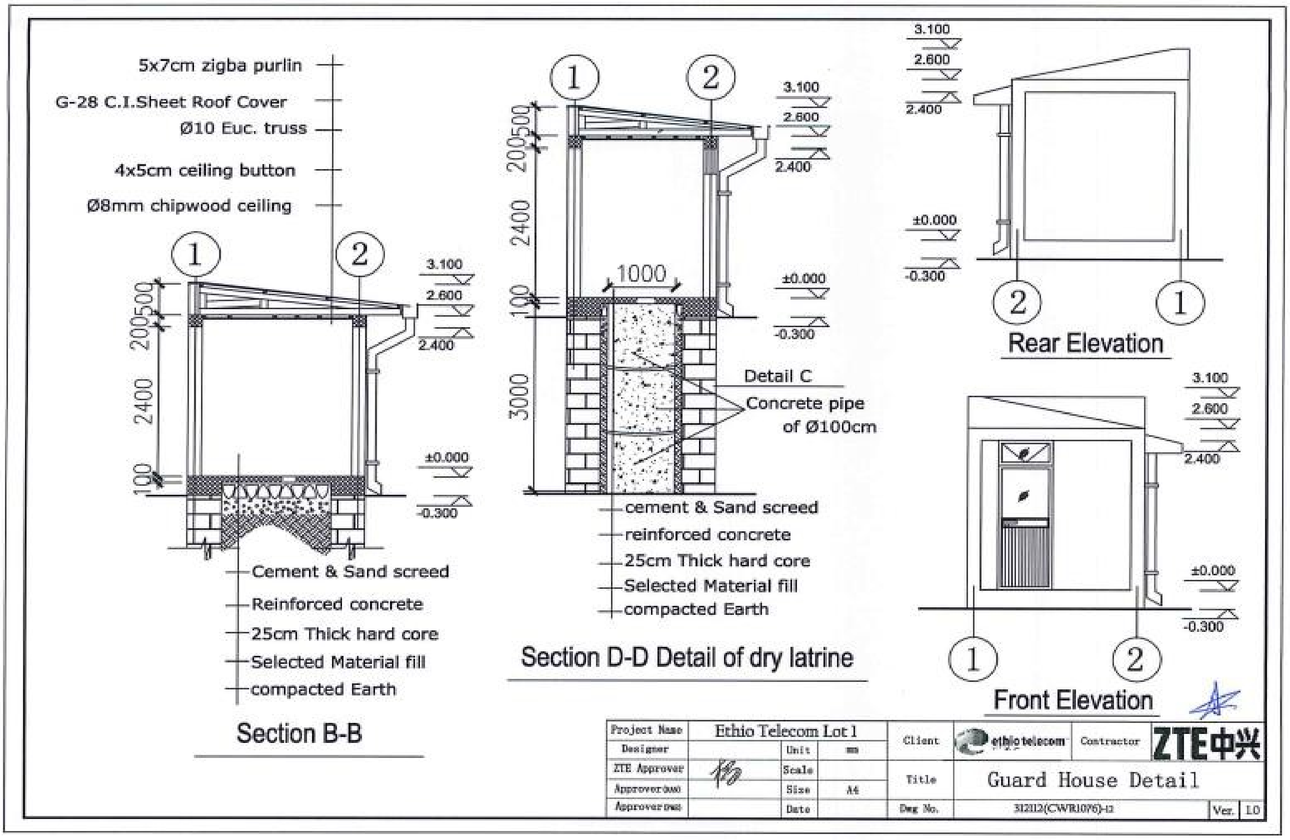
Concrete slab							
Type	B(mm)	D(mm)	Slab Thickness(mm)	Qty.	Concrete Volume(m3)		
slab	3000	1400	400	1	1.68		
Plain concrete	3200	1600	100	1	0.51		
Rebar							
Bar mark	Shape	Bar Size(mm)	L(mm)	Bar Length(mm)	Qty.	Total Rebar Mass(kg)	Total Rebar Mass(kg)
①	300 L 300	12	2900	3500	16	49.73	103.72
②		12	1300	1900	32	53.99	

Notes

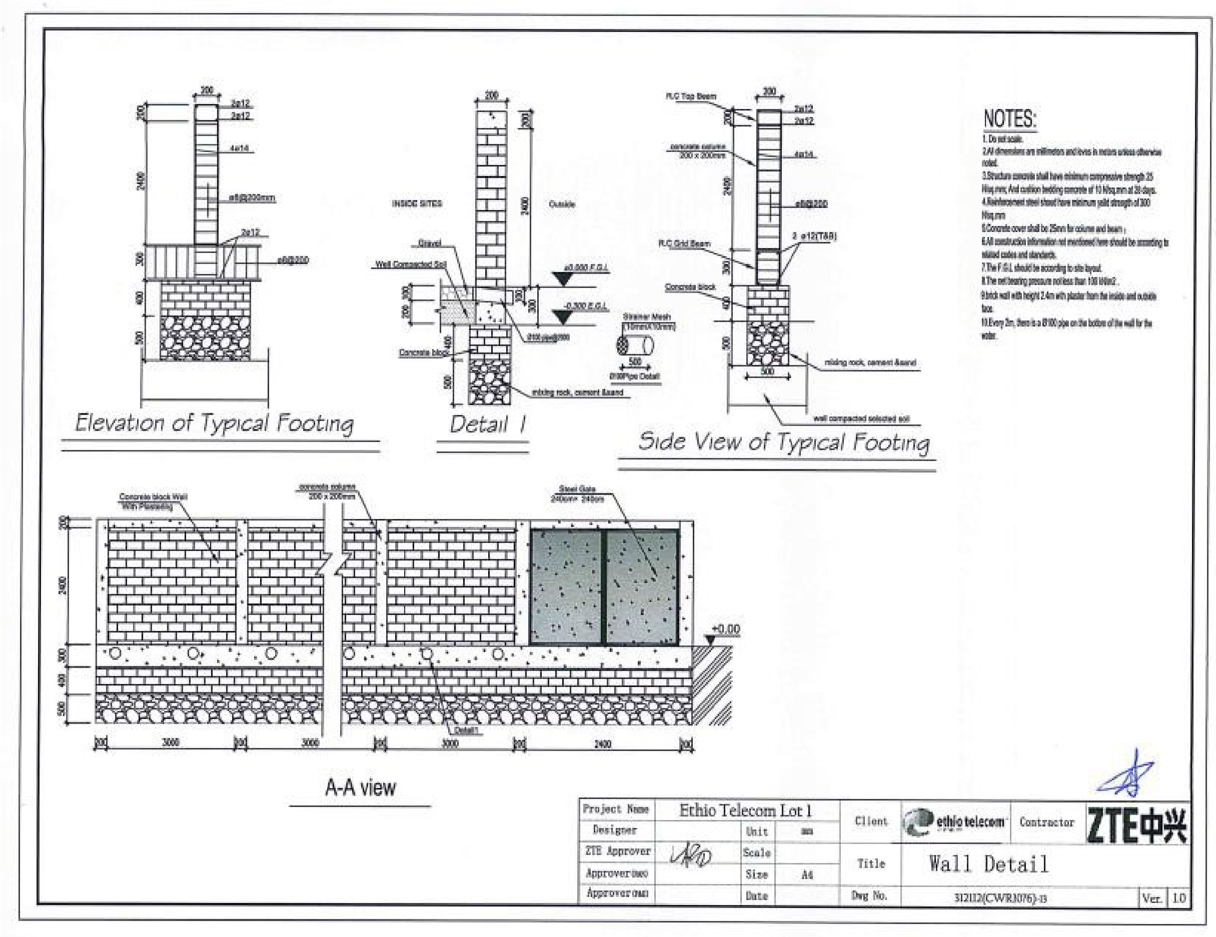
1.All dementions are shown in millimeters(mm),unless otherwise noted ;
Elevation shall be shown in meters(m) unless otherwise noted ;
2.The strength of foundation concrete shall be 3500psi(24.08Mpa);bed course concrete strengh shall be 2500psi(17.2Mpa);
3.Reinforcement for foundation shall be grade 300Mpa,Ring reinforcement shall be grade 280Mpa(275.8Mpa);
4.Concrete cover to reinforcement shall be 50mm in case of concrete bed course existing,otherwise 70mm ;
5.The foundation shall loacte on solid soil;otherwise put good grades gravel under foundation.

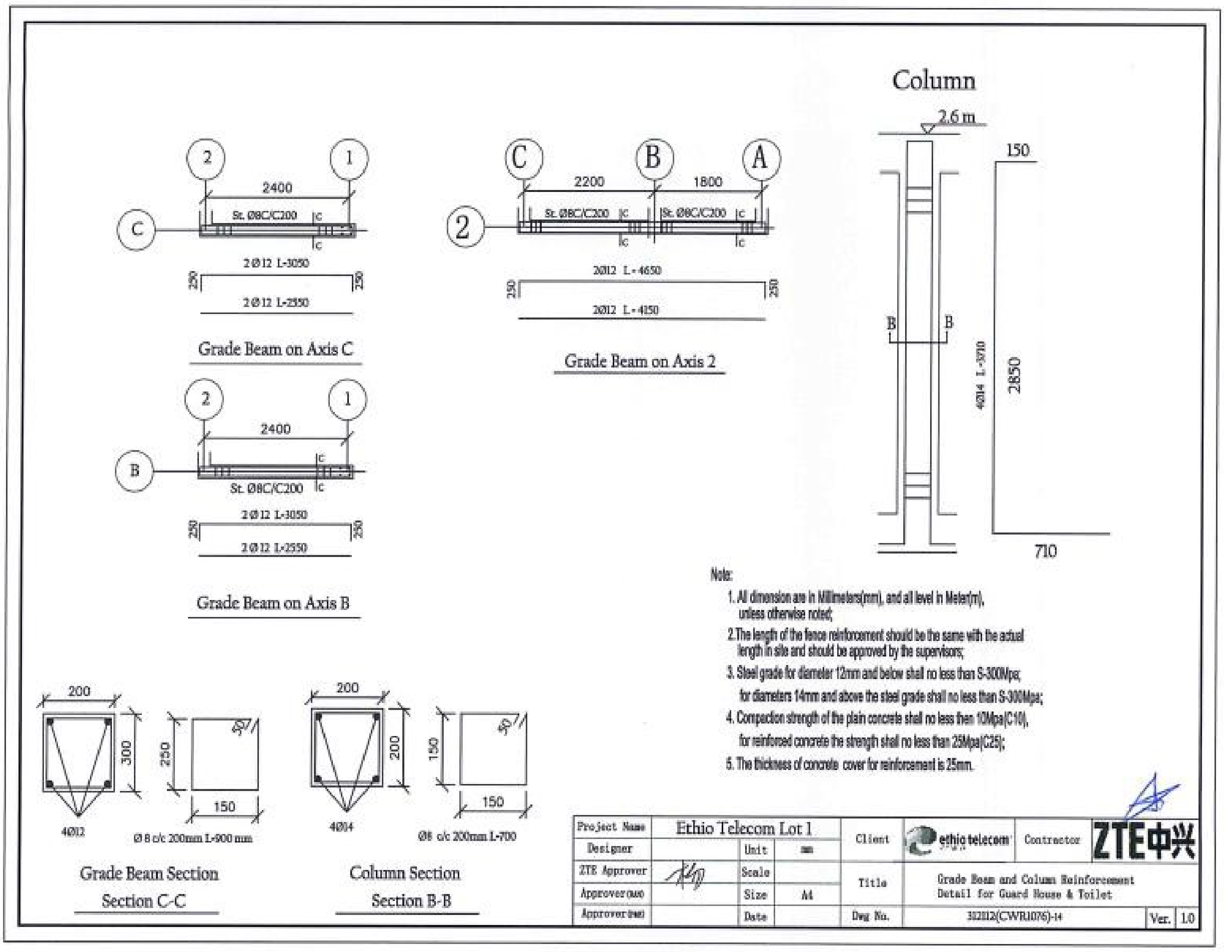
Project Name	Ethio Telecom Lot 1			Client	ethio telecom	Contractor	ZTE中兴
Designer		Unit	mm				
ZTE Approver		Scale		Title	Outdoor Equipment Foundation		
Approver		Size	A4				
Approver		Date		Dwg No.	312112(CWR1076)-10	Ver.	1.0


Boundary Wall Layout
Left Side Elevation
Right Side Elevation
Parapet HCB wall
If the last section is shorter than 3m, divided it into two equal parts and add each part to previous two sections.
F.G.L(±0.000)
E.G.L(-0.300)
Guard house
Dry latrine
Detail 1
Detail C
Wire mesh
Project Name
Ethio Telecom Lot 1
Designer
ZTE Approver
Unit
Scale
Size
Date
Client
Contractor
Title
Boundary Wall Layout
Dwg No.
Ver. 1.0

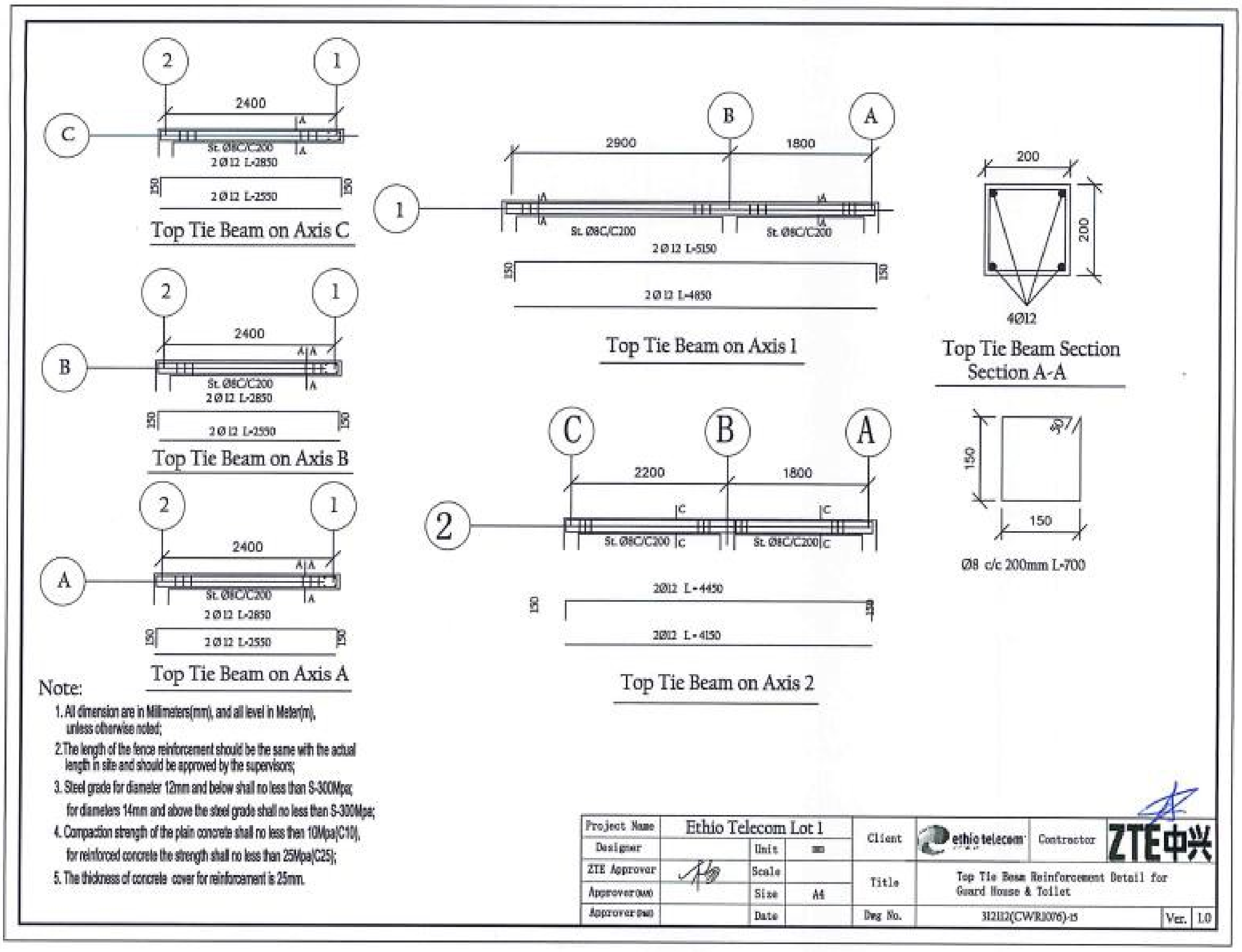


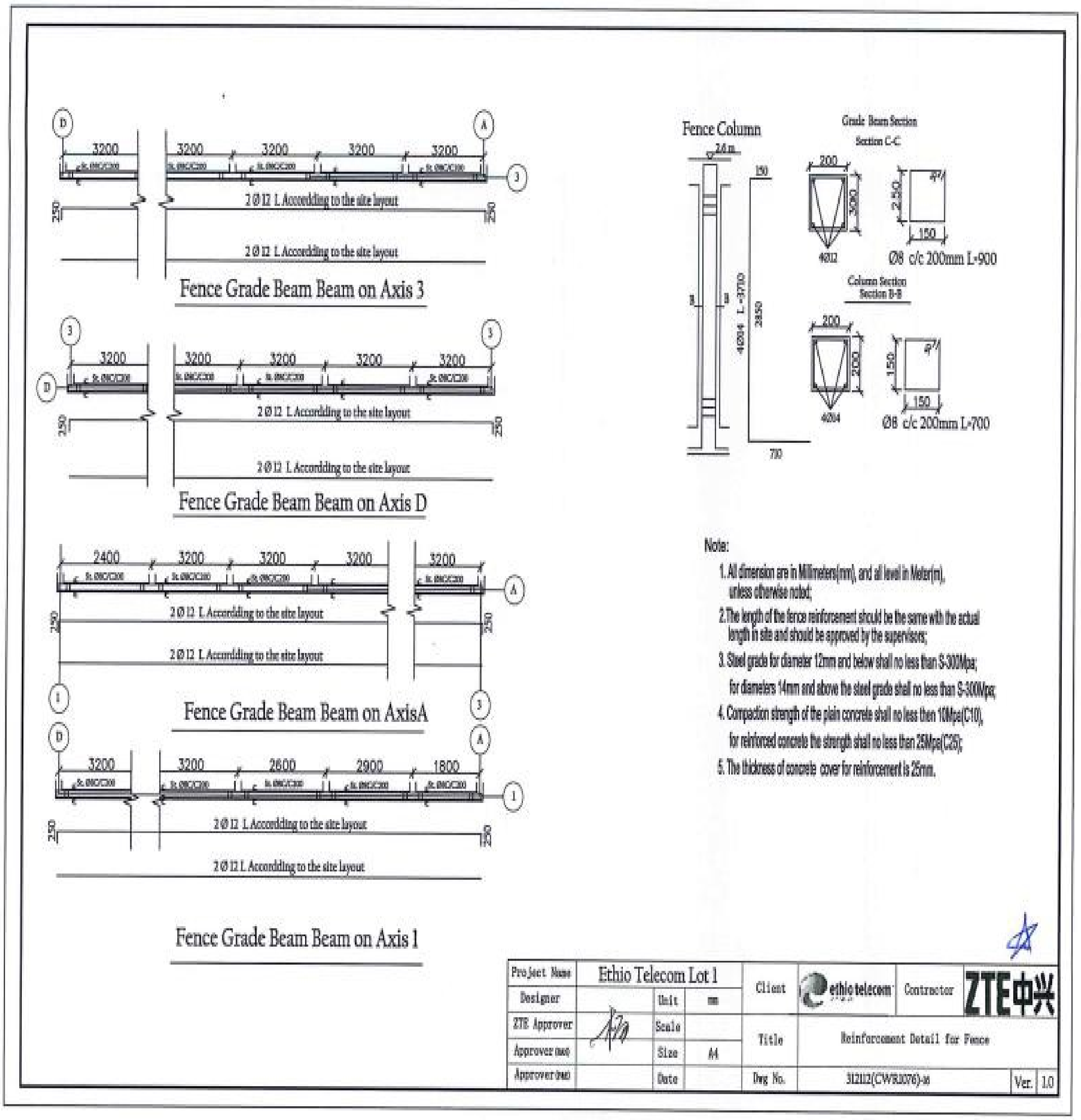
5x7cm zigba purlin
G-28 C.I.Sheet Roof Cover
Ø10 Euc. truss
4x5cm ceiling button
Ø8mm chipwood ceiling
Cement & Sand screed
Reinforced concrete
25cm Thick hard core
Selected Material fill
compacted Earth
Section B-B
Detail C
Concrete pipe of Ø100cm
cement & Sand screed
reinforced concrete
25cm Thick hard core
Selected Material fill
compacted Earth
Section D-D Detail of dry latrine
Rear Elevation
Front Elevation
Project Name
Ethio Telecom Lot 1
Designer
Unit
mm
ZTE Approver
Scale
Size
A4
Date
Client
ethio telecom
Contractor
ZTE中兴
Title
Guard House Detail
Dwg No.
Ver.
1.0


Elevation of Typical Footing
Detail I
Side View of Typical Footing
NOTES:
A-A view
Project Name
Ethio Telecom Lot 1
Designer
Unit
ZTE Approver
Scale
Size
A4
Date
Client
ethio telecom
Contractor
ZTE中兴
Title
Wall Detail
Dwg No.
Ver.
1.0

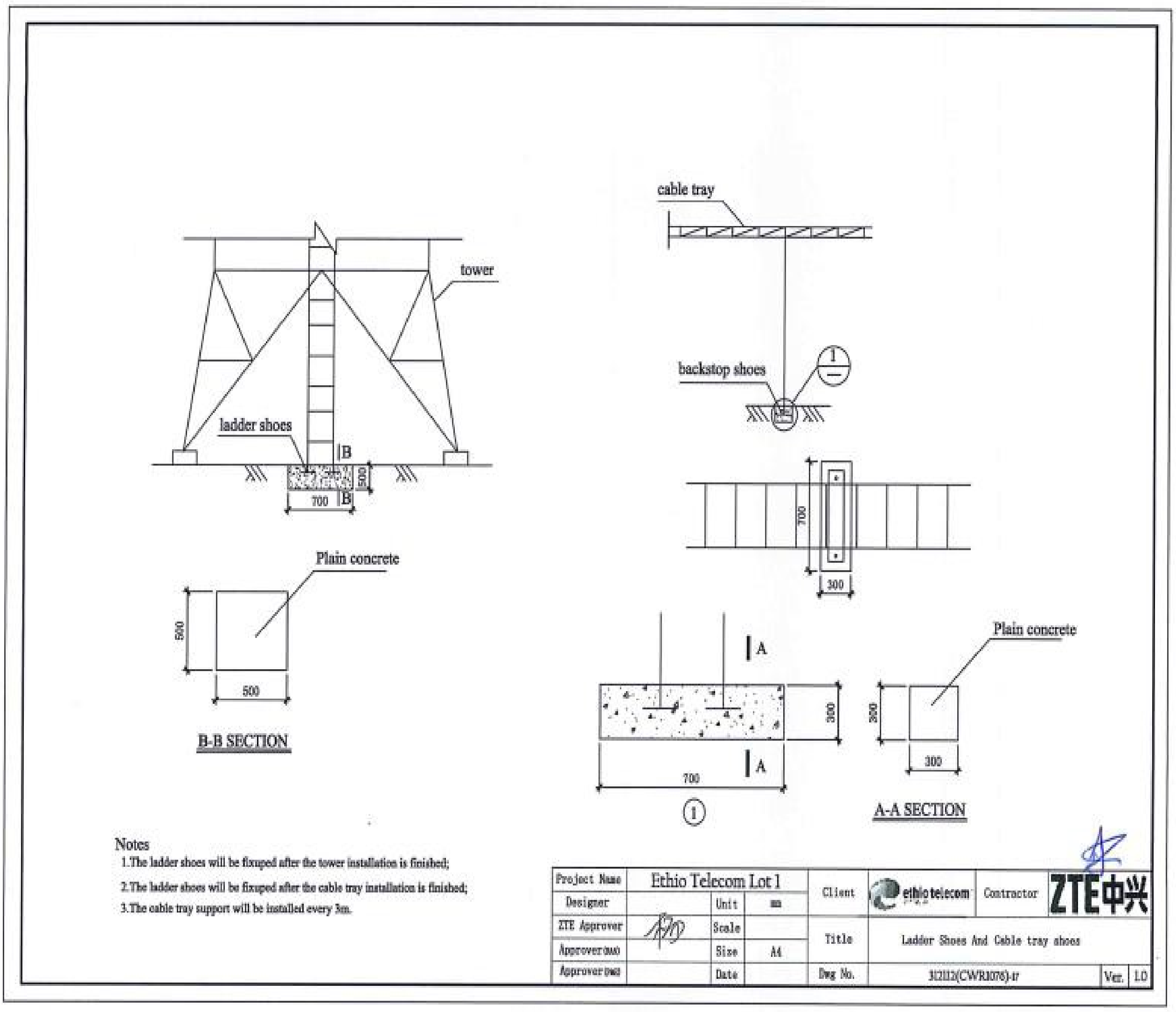


Column
2.6 m
150
2850
710
Grade Beam on Axis C
Grade Beam on Axis 2
Grade Beam on Axis B
2400
2200
1800
St. Ø8C/C200
2Ø12 L-3050
2Ø12 L-2550
2Ø12 L-4650
2Ø12 L-4150
Note:
1. All dimension are in Millimeters(mm), and all level in Meter(m), unless otherwise noted;
2. The length of the fence reinforcement should be the same with the actual length in site and should be approved by the supervisors;
3. Steel grade for diameter 12mm and below shall no less than S-300Mpa; for diameters 14mm and above the steel grade shall no less than S-300Mpa;
4. Compaction strength of the plain concrete shall no less then 10Mpa(C10), for reinforced concrete the strength shall no less than 25Mpa(C25);
5. The thickness of concrete cover for reinforcement is 25mm.
4Ø12
Ø 8 c/c 200mm L-900 mm
4Ø14
Ø8 c/c 200mm L-700
Grade Beam Section
Section C-C
Column Section
Section B-B
Project Name
Ethio Telecom Lot 1
Designer
ZTE Approver
Unit
mm
Scale
Size
A4
Date
Client
ethio telecom
Contractor
ZTE中兴
Title
Grade Beam and Column Reinforcement Detail for Guard House & Toilet
Dwg No.
Ver.
1.0


Top Tie Beam on Axis C
Top Tie Beam on Axis B
Top Tie Beam on Axis A
Top Tie Beam on Axis 1
Top Tie Beam on Axis 2
Top Tie Beam Section
Section A-A
Ø8 c/c 200mm L-700
Note:
1. All dimension are in Millimeters(mm), and all level in Meter(m), unless otherwise noted;
2. The length of the fence reinforcement should be the same with the actual length in site and should be approved by the supervisors;
3. Steel grade for diameter 12mm and below shall no less than S-300Mpa; for diameters 14mm and above the steel grade shall no less than S-300Mpa;
4. Compaction strength of the plain concrete shall no less then 10Mpa(C10), for reinforced concrete the strength shall no less than 25Mpa(C25);
5. The thickness of concrete cover for reinforcement is 25mm.
Project Name
Ethio Telecom Lot 1
Client
ethio telecom
Contractor
Title
Top Tie Beam Reinforcement Detail for Guard House & Toilet
Size
A4
Dwg No.
312112(CWR1076)-15



Fence Grade Beam Beam on Axis 3
2 Ø 12 L Accordding to the site layout
Fence Grade Beam Beam on Axis D
Fence Grade Beam Beam on AxisA
Fence Grade Beam Beam on Axis 1
Fence Column
Grade Beam Section
Section C-C
Ø8 c/c 200mm L=900
Column Section
Section B-B
Ø8 c/c 200mm L=700
Note:
1. All dimension are in Millimeters(mm), and all level in Meter(m), unless otherwise noted;
2. The length of the fence reinforcement should be the same with the actual length in site and should be approved by the supervisors;
3. Steel grade for diameter 12mm and below shall no less than S-300Mpa; for diameters 14mm and above the steel grade shall no less than S-300Mpa;
4. Compaction strength of the plain concrete shall no less then 10Mpa(C10), for reinforced concrete the strength shall no less than 25Mpa(C25);
5. The thickness of concrete cover for reinforcement is 25mm.
Project Name
Ethio Telecom Lot 1
Client
Contractor
Designer
Unit
mm
ZTE Approver
Scale
Title
Reinforcement Detail for Fence
Size
A4
Date
Dwg No.
Ver.
1.0


cable tray
tower
backstop shoes
ladder shoes
B
700
500
Plain concrete
500
500
B-B SECTION
700
300
A
A
700
300
300
300
Plain concrete
A-A SECTION
Notes
1.The ladder shoes will be fixuped after the tower installation is finished;
2.The ladder shoes will be fixuped after the cable tray installation is finished;
3.The cable tray support will be installed every 3m.
Project Name
Ethio Telecom Lot 1
Client
ethio telecom
Contractor
ZTE中兴
Designer
Unit
mm
ZTE Approver
Scale
Title
Ladder Shoes And Cable tray shoes
Size
A4
Date
Dwg No.
Ver.
1.0

SAP2000 Analysis Report

Prepared by
DiLshad Sys

Model Name: mignot model1.SDB

8 May 2016

1. Model geometry

This section provides model geometry information, including items such as joint coordinates, joint restraints, and element connectivity.

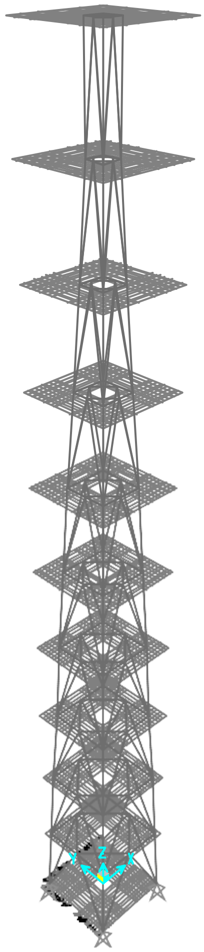


Figure 1: Finite element model

1.1. Joint coordinates

Table 1: Joint Coordinates

Table 1: Joint Coordinates

Joint	CoordSys	CoordType	GlobalX	GlobalY	GlobalZ
			m	m	m
1	GLOBAL	Cartesian	-2.55000	-2.55000	0.00000
2	GLOBAL	Cartesian	-2.32500	-2.32500	5.00000
3	GLOBAL	Cartesian	-2.32500	0.00000	5.00000
4	GLOBAL	Cartesian	0.00000	-2.32500	5.00000
5	GLOBAL	Cartesian	-2.55000	2.55000	0.00000

1.2. Joint restraints

Table 2: Joint Restraint Assignments

Table 2: Joint Restraint Assignments

Joint	U1	U2	U3	R1	R2	R3
1	Yes	Yes	Yes	No	No	No

5	Yes	Yes	Yes	No	No	No

Table 2: Joint Restraint Assignments

Joint	U1	U2	U3	R1	R2	R3
8	Yes	Yes	Yes	No	No	No
11	Yes	Yes	Yes	No	No	No

1.3. Element connectivity

Table 3: Connectivity - Frame

Table 3: Connectivity - Frame

Frame	JointI	JointJ	Length
			m
1	1	2	5.01011
2	1	3	5.61722
3	1	4	5.61722
4	5	6	5.01011
5	5	3	5.61722

Table 4: Frame Section Assignments

Table 4: Frame Section Assignments

Frame	AnalSect	DesignSect	MatProp
1	L4X3X1/2	L5X5X5/8	Default
2	L4X3X1/2	L2X2X1/4	Default
3	L4X3X1/2	L2X2X1/4	Default
4	L4X3X1/2	L5X5X3/4	Default
5	L4X3X1/2	L2X2X1/4	Default

Table 5: Frame Release Assignments 1 - General, Part 1 of 2

Table 5: Frame Release Assignments 1 - General, Part 1 of 2

Frame	PI	V2I	V3I	TI	M2I	M3I
2	No	No	No	Yes	Yes	Yes
3	No	No	No	Yes	Yes	Yes
5	No	No	No	Yes	Yes	Yes
6	No	No	No	Yes	Yes	Yes
8	No	No	No	Yes	Yes	Yes
9	No	No	No	Yes	Yes	Yes
11	No	No	No	Yes	Yes	Yes
12	No	No	No	Yes	Yes	Yes

2. Material properties

This section provides material property information for materials used in the model.

Table 6: Material Properties 02 - Basic Mechanical Properties

Table 6: Material Properties 02 - Basic Mechanical Properties

Material	UnitWeight	UnitMass	E1	G12	U12	A1
	KN/m3	KN-s2/m4	KN/m2	KN/m2		1/C
4000Psi	2.3563E+01	2.4028E+00	24855578.28	10356490.95	0.200000	9.9000E-06

Table 6: Material Properties 02 - Basic Mechanical Properties

Material	UnitWeight	UnitMass	E1	G12	U12	A1
	KN/m3	KN-s2/m4	KN/m2	KN/m2		1/C
A992Fy50	7.6973E+01	7.8490E+00	199947978.8	76903068.77	0.300000	1.1700E-05

Table 7: Material Properties 03a - Steel Data

Table 7: Material Properties 03a - Steel Data

Material	Fy	Fu	FinalSlope
	KN/m2	KN/m2	
4000Psi	248211.28	399895.96	-0.100000
A992Fy50	344737.89	448159.26	-0.100000

3. Section properties

This section provides section property information for objects used in the model.

3.1. Frames

Table 8: Frame Section Properties 01 - General, Part 1 of 4

Table 8: Frame Section Properties 01 - General, Part 1 of 4

SectionName	Material	Shape	t3	t2	tf	tw	t2b	tfb
			m	m	m	m	m	m
AUTO1		Auto Select						
FSEC1	A992Fy50	I/Wide Flange	0.304800	0.127000	0.009652	0.006350	0.127000	0.009652
L2-1/2X2-1/2X1/2	A992Fy50	Angle	0.063500	0.063500	0.012700	0.012700		
L2-1/2X2-1/2X1/4	A992Fy50	Angle	0.063500	0.063500	0.006350	0.006350		
L2-1/2X2-1/2X3/16	A992Fy50	Angle	0.063500	0.063500	0.004763	0.004763		
L2-1/2X2-1/2X3/8	A992Fy50	Angle	0.063500	0.063500	0.009525	0.009525		

Table 8: Frame Section Properties 01 - General, Part 2 of 4

Table 8: Frame Section Properties 01 - General, Part 2 of 4

SectionName	Area	TorsConst	I33	I22	AS2	AS3
	m2	m4	m4	m4	m2	m2
AUTO1						
FSEC1	0.004265	9.651E-08	0.000066	3.301E-06	0.001935	0.002043
L2-1/2X2-1/2X1/2	0.001452	7.825E-08	5.078E-07	5.078E-07	0.000806	0.000806

L4X3X1/4	0.000016	9.580E-06	0.000029	0.000017	0.032401	0.022533

Table 8: Frame Section Properties 01 - General, Part 4 of 4

Table 8: Frame Section Properties 01 - General, Part 4 of 4

SectionName	AMod	A2Mod	A3Mod	JMod	I2Mod	I3Mod	MMod	WMod
AUTO1								
FSEC1	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
L2-1/2X2-1/2X1/2	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
L2-1/2X2-1/2X1/4	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000

4. Load patterns

This section provides loading information as applied to the model.

4.1. Definitions

Table 9: Load Pattern Definitions

Table 9: Load Pattern Definitions

LoadPat	DesignType	SelfWtMult	AutoLoad
DEAD	DEAD	1.000000	
EQx	QUAKE	0.000000	EUROCODE8 2004
EQy	QUAKE	0.000000	EUROCODE8 2004
w ind load	WIND	0.000000	EUROCODE1 2005

4.2. Auto wind loading

Table 10: Auto Wind - Eurocode1 2005, Part 1 of 2

Table 10: Auto Wind - Eurocode1 2005, Part 1 of 2

LoadPat	Angle	Windw ardCp	LeewardCp	MaxZ	MinZ
	Degrees			m	m
w ind load	0.000	0.800000	0.500000	50.00000	0.00000

Table 10: Auto Wind - Eurocode1 2005, Part 2 of 2

Table 10: Auto Wind - Eurocode1 2005, Part 2 of 2

LoadPat	WindSpeed	TerrainCat	OroFact	TurbFact	StructFact	AirDensity
	meter/sec					
w ind load	22.000	II	1.000000	1.000000	1.000000	1.250000

4.3. Auto seismic loading

Table 11: Auto Seismic - Eurocode8 2004, Part 1 of 3

Table 11: Auto Seismic - Eurocode8 2004, Part 1 of 3

LoadPat	Dir	PercentEcc	MaxZ	MinZ	Country	Ag
			m	m		
EQx	X	0.050000	50.00000	0.00000	Other	0.100000
EQy	Y	0.050000	50.00000	0.00000	Other	0.100000

Table 11: Auto Seismic - Eurocode8 2004, Part 2 of 3

Table 11: Auto Seismic - Eurocode8 2004, Part 2 of 3

LoadPat	SpecType	GroundType	SoilFact	Tb	Tc	Td	LBFact	BehaveFact
				Sec	Sec	Sec		
EQx	1	B	1.200000	0.1500	0.5000	2.0000	0.200000	2.000000
EQy	1	B	1.200000	0.1500	0.5000	2.0000	0.200000	2.000000

Table 11: Auto Seismic - Eurocode8 2004, Part 3 of 3

Table 11: Auto Seismic - Eurocode8 2004, Part 3 of 3

LoadPat	CorrFact	TUsed	CoeffUsed	WeightUsed	BaseShear
		Sec		KN	KN
EQx	1.000000	1.0000	0.075000	129.774	9.733
EQy	1.000000	1.0000	0.075000	129.774	9.733

5. Load cases

This section provides load case information.

5.1. Definitions

Table 12: Load Case Definitions

Table 12: Load Case Definitions

Case	Type	InitialCond	ModalCase	BaseCase
DEAD	LinStatic	Zero		
MODAL	LinModal	Zero		
COMB1-NL	NonStatic	Zero		
EQx	LinStatic	Zero		
EQy	LinStatic	Zero		
w ind load	LinStatic	Zero		

5.2. Static case load assignments

Table 13: Case - Static 1 - Load Assignments

Table 13: Case - Static 1 - Load Assignments

Case	LoadType	LoadName	LoadSF
DEAD	Load pattern	DEAD	1.000000
COMB1-NL	Load pattern	DEAD	1.200000
EQx	Load pattern	EQx	1.000000
EQy	Load pattern	EQy	1.000000
w ind load	Load pattern	w ind load	1.000000

5.3. Response spectrum case load assignments

Table 14: Function - Response Spectrum - User

Table 14: Function - Response Spectrum - User

Name	Period Sec	Accel	FuncDamp
UNIFRS	0.000000	1.000000	0.050000
UNIFRS	1.000000	1.000000	

6. Load combinations

This section provides load combination information.

Table 15: Combination Definitions

Table 15: Combination Definitions

ComboName	ComboType	CaseName	ScaleFactor
combo 1	Linear Add	DEAD	1.200000
combo 1		w ind load	1.600000
combo2	Linear Add	DEAD	0.900000
combo2		w ind load	1.600000
combo3	Linear Add	DEAD	1.200000
combo3		EQx	-1.000000
combo4	Linear Add	DEAD	1.200000

7. Structure results

This section provides structure results, including items such as structural periods and base reactions.

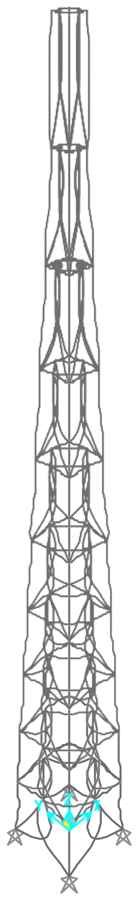


Figure 2: Deformed shape

7.1. Mass summary

Table 16: Assembled Joint Masses

Base

Table 16: Assembled Joint Masses

Joint	U1 KN-s2/m	U2 KN-s2/m	U3 KN-s2/m	R1 KN-m-s2	R2 KN-m-s2	R3 KN-m-s2
1	0.13	0.13	0.13	0.0000	0.0000	0.0000
2	0.21	0.21	0.21	0.0000	0.0000	0.0000
3	0.18	0.18	0.18	0.0000	0.0000	0.0000
4	0.18	0.18	0.18	0.0000	0.0000	0.0000
5	0.13	0.13	0.13	0.0000	0.0000	0.0000

reactions

Table 17: Base Reactions

Table 17: Base Reactions

OutputCase	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m
DEAD	-2.331E-15	1.556E-14	129.774	-1.007E-12	-3.286E-13	3.020E-14
EQx	-9.733	2.215E-12	1.723E-13	-9.643E-11	-315.5021	4.963E-12
EQy	2.210E-12	-9.733	-4.860E-13	315.5021	9.624E-11	-2.713E-12
w ind load	0.000	0.000	0.000	0.0000	0.0000	0.0000

8. Joint results

This section provides joint results, including items such as displacements and reactions.

Table 18: Joint Displacements

Table 18: Joint Displacements

Joint	OutputCase	U1 m	U2 m	U3 m	R1 Radians	R2 Radians	R3 Radians
1	DEAD	0.000000	0.000000	0.000000	-0.000079	0.000079	-1.326E-19
1	EQx	0.000000	0.000000	0.000000	0.000065	-0.000026	-3.962E-06
1	EQy	0.000000	0.000000	0.000000	0.000026	-0.000065	3.962E-06
1	w ind load	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
2	DEAD	-1.735E-06	-1.735E-06	-0.000349	0.000059	-0.000059	-1.508E-19
2	EQx	0.000081	1.980E-07	0.000340	-0.000122	0.000090	-7.812E-07

Table 20: Element Forces - Frames, Part 2 of 2

Table 20: Element Forces - Frames, Part 2 of 2

Frame	Station m	OutputCase	T KN-m	M2 KN-m	M3 KN-m

10.Material take-off

This section provides a material take-off.

Table 21: Material List 2 - By Section Property

Table 21: Material List 2 - By Section Property

Section	ObjectType	Num Pieces	TotalLength m	TotalWeight KN
L4X3X1/2	Frame	200	804.07792	129.774

11. Design preferences

This section provides the design preferences for each type of design, which typically include material reduction factors, framing type, stress ratio limit, deflection limits, and other code specific items.

11.1. Steel design

Table 22: Preferences - Steel Design - Eurocode 3-2005, Part 1 of 2

Table 22: Preferences - Steel Design - Eurocode 3-2005, Part 1 of 2

FrameType	PatLLF	SRatioLimit	Country	CombosEq	KFactorMethod
Moment Frame	0.750000	0.950000	CEN Default	Eq. 6.10	Method 2 (Annex B)

Table 22: Preferences - Steel Design - Eurocode 3-2005, Part 2 of 2

Table 22: Preferences - Steel Design - Eurocode 3-2005, Part 2 of 2

GammaM0	GammaM1	GammaM2	DLRat	SDLAndLLRat	LLRat	TotalRat	NetRat
1.000000	1.000000	1.250000	120.000000	120.000000	360.000000	240.000000	240.000000

11.2. Concrete design

Table 23: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 1 of 2

Table 23: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 1 of 2

MinEccen	PatLLF	UFLimit	SeisCat	PhiT
No	0.750000	0.950000	D	0.900000

Table 23: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 2 of 2

Table 23: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 2 of

PhiCTied	PhiCSpiral	PhiV	PhiVSeismic	PhiVJoint
0.650000	0.700000	0.750000	0.600000	0.850000

11.4. Cold formed design

Table 25: Preferences - Cold Formed Design - AISI-ASD96

Table 25: Preferences - Cold Formed Design - AISI-ASD96

FrameType	SRatioLimit	OmegaBS	OmegaBUS	OmegaBLTB	OmegaVS	OmegaVNS	OmegaT	OmegaC
Braced Frame	1.000000	1.670000	1.670000	1.670000	1.670000	1.500000	1.670000	1.800000

12. Design overwrites

This section provides the design overwrites for each type of design, which are assigned to individual members of the structure.

12.1. Steel design

Table 26: Overwrites - Steel Design - Eurocode 3-2005, Part 1 of 5

Table 26: Overwrites - Steel Design - Eurocode 3-2005, Part 1 of 5

Frame	DesignSect	FrameType	Fy KN/m2	RLLF	AreaRatio	XLMajor	XLMinor
1	Program Determined	Program Determined	0.00	0.000000	0.000000	0.000000	0.000000
2	Program Determined	Program Determined	0.00	0.000000	0.000000	0.000000	0.000000
3	Program Determined	Program Determined	0.00	0.000000	0.000000	0.000000	0.000000

13.

Design summary

This section provides the design summary for each type of design, which highlights the controlling demand/capacity ratio and it's associated combination and location in each member.

13.1. Steel design

Table 27: Steel Design 1 - Summary Data - Eurocode 3-2005, Part 1 of 2

Table 27: Steel Design 1 - Summary Data - Eurocode 3-2005, Part 1 of 2

Frame	Location m	Combo	DesignSect	DesignType	Ratio	RatioType
1	0.00000	DSTL12	L5X5X5/8	Column	0.945784	PMM
2	2.80861	DSTL12	L2X2X1/4	Brace	0.659567	PMM
3	2.80861	DSTL10	L2X2X1/4	Brace	0.659567	PMM
4	0.00000	combo5	L5X5X3/4	Column	0.851790	PMM

APPENDIX C# PROGRAM

```
using System;
using System.Collections.Generic;
using System.ComponentModel;
using System.Data;
using System.Drawing;
using System.Linq;
using System.Text;
using System.Threading.Tasks;
using System.Windows.Forms;
using System.Windows.Forms.DataVisualization.Charting;

namespace deflection_and_rotation_calculater
{
    public partial class Form1 : Form
    {
        public Form1()
        {
            InitializeComponent();
        }

        private void Calculate_Click(object sender, EventArgs e)
        {
          // DEFINE VARIABLES

            Int32 length;
            Double b;
```

```
Double   o;
Double    l;
Double   x;
Double   n;
Double  p;


Double  n4;
Double  n5;
Double  n6;
   {
      textboxdeflection.Text  = Convert.ToString("Error!!!  you inserted distance x out of the beam length");


   }


}
else if (n>19 && n <= 20)
{




   if (x <= s)
   {
      rotationAtx  = moverEIatn1 * x;
      textboxrotation.Text  = Convert.ToString(rotationAtx);
   }
   else
      if (x > s && x <= 2 * s)
      {
```

```
}
else
    if (x > 2 * s && x <= s * 3)
    {
        rotationAtx = moverEIatn1 * s +
                      moverEIatn2 * s +
                      moverEIatn3 * (x - (2 * s));

        textboxrotation.Text  = Convert.ToString(rotationAtx);
    }

    else if (x > 3 * s && x <= 4 * s)
    {



    else if (x > 4 * s && x <= 5 * s)
    {
        rotationAtx = moverEIatn1 * s +
           moverEIatn2 * s +
           moverEIatn3 * s +
           moverEIatn4 * s +
           moverEIatn5 * (x - (4 * s));
        textboxrotation.Text  = Convert.ToString(rotationAtx);
    }
    else if (x > 5 * s && x <= 6 * s)
    {
    }
    else if (x > 6 * s && x <= 7 * s)
```

```
{
   rotationAtx = moverEIatn1 * s +
      moverEIatn2 * s +
      moverEIatn3 * s +
      moverEIatn4 * s +
      moverEIatn5 * s +
      moverEIatn6 * s +
      moverEIatn7 * (x - (6 * s));
   textboxrotation.Text  = Convert.ToString(rotationAtx);
}
else if (x > 7 * s && x <= 8 * s)
{
   rotationAtx = moverEIatn1 * s +
      moverEIatn2 * s +
      moverEIatn3 * s +
      moverEIatn4 * s +
}
else if (x > 8 * s && x <= 9 * s)
{
   rotationAtx = moverEIatn1 * s +
      moverEIatn2 * s +
      moverEIatn3 * s +
      moverEIatn4 * s +

   textboxrotation.Text  = Convert.ToString(rotationAtx);
}

else if (x > 9 * s && x <= 10 * s)
```

```
{
    rotationAtx = moverEIatn1 * s +
        moverEIatn2 * s +
        moverEIatn3 * s +
        moverEIatn4 * s +
}



        else
        {
            textboxrotation.Text = Convert.ToString("Error!!! you inserted distance x out of the beam length");

        }



if (x <= s)
{
    deflectionAtx = moverEIatn1 * x * (x / 2);
    textboxdeflection.Text = Convert.ToString(deflectionAtx);
}
else if (x > s && x <= 2 * s)
{
    deflectionAtx = moverEIatn1 * s * (x - (s / 2)) +
        moverEIatn2 * (x - s) * ((x - s) / 2);
    textboxdeflection.Text = Convert.ToString(deflectionAtx);
```

```
}

  textboxdeflection.Text  =  Convert.ToString(deflectionAtx);

}
else if (x > 6 * s && x <= 7 * s)
{
  deflectionAtx = moverEIatn1 * s * (x - (s / 2)) +
     moverEIatn2 * s * (x - (s + (s / 2))) +
     moverEIatn3 * s * (x - ((2 * s) + (s / 2))) +
     moverEIatn4 * s * (x - ((3 * s) + (s / 2))) +
     moverEIatn5 * s * (x - ((4 * s) + (s / 2))) +
     moverEIatn6 * s * (x - ((5 * s) + (s / 2))) +
     moverEIatn7 * (x - (6 * s)) * ((x - (6 * s)) / 2);

  textboxdeflection.Text  =  Convert.ToString(deflectionAtx);

}
else if (x > 7 * s && x <= 8 * s)
{
  deflectionAtx = moverEIatn1 * s * (x - (s / 2)) +
     moverEIatn2 * s * (x - (s + (s / 2))) +
     moverEIatn3 * s * (x - ((2 * s) + (s / 2))) +
     moverEIatn4 * s * (x - ((3 * s) + (s / 2))) +

  textboxdeflection.Text  =  Convert.ToString(deflectionAtx);

}
```

```
        {
            textboxdeflection.Text  = Convert.ToString("Error!!!  you inserted distance x out of the beam length");
        }


    }
else if (n>99 && n <= 100)
    {
        if (x <= s)


            }
            else
                if (x > 2 * s && x <= s * 3)
                {
                    rotationAtx = moverEIatn1 * s +
                                  moverEIatn2 * s +
                                  moverEIatn3 * (x - (2 * s));


                    textboxrotation.Text  = Convert.ToString(rotationAtx);
                }


                else if (x > 3 * s && x <= 4 * s)
```